PROGRAM MANAGER
FOR ROCKY MOUNTAIN ARSENAL

U.S. ARMY MATERIEL COMMAND

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**Final Work Plan for Material
and Area Feasibility Studies
Soils Support Program
Rocky Mountain Arsenal
Commerce City, Colorado**

**Rocky Mountain Arsenal
Information Center
Commerce City, Colorado**

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Harding Lawson Associates

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THE PURPOSE OF THIS WORK PLAN IS TO OUTLINE THE PROCEDURES AND LOGIC USED TO CONDUCT MATERIAL AND AREA FEASIBILITY SUBTASKS FOR THE FEASIBILITY STUDY (FS) SOILS SUPPORT PROGRAM. THIS WORK PLAN IS ORGANIZED TO PRESENT THE REQUISITE TASK BACKGROUND AND OBJECTIVES; DESCRIBES EACH SUBTASK'S DESIGN; PROVIDE CONSTRUCTION, DRILLING, AND SAMPLING PROCEDURES FOR THE FIELD PROGRAMS; PRESENT PROPOSED GEOTECHNICAL TESTING METHODS; AND DISCUSS DATA EVALUATION AND REPORTING PROCEDURES AND PROTOCOLS. SECTION 1.2 OF THIS DOCUMENT PRESENTS TASK BACKGROUND AND OBJECTIVES; SECTION 2.0 PRESENTS TEST FILL CONSTRUCTION, TESTING PROCEDURES, AND DATA EVALUATION, AND SECTION 3.0 PRESENTS GEOLOGIC AND GEOTECHNICAL SUBTASK DESIGN, PROCEDURES, GEOTECHNICAL TESTING, AND DATA EVALUATION.

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November 9, 1994

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CONTENTS

1.0 INTRODUCTION

This Work Plan was prepared for the Material and Area Feasibility Studies portion of the Feasibility Study (FS) Soils Support Program as Data Requirement A002, a contract deliverable under Delivery Order 0007 (Modification to Task 93-03 Feasibility Study Soils Support Program) of Contract DAAA05-92-D-0003 between Harding Lawson Associates (HLA) and the U.S. Department of the Army (Army). This report was prepared by HLA at the direction of the Army for the sole use of the Army and the signatories of the Federal Facilities Agreement (FFA) of Rocky Mountain Arsenal (RMA), the only intended beneficiaries of this work.

1.1 Document Purpose, Scope, and Organization

The purpose of this Work Plan is to outline the procedures and logic used to conduct the Material and Area Feasibility Subtasks for the FS Soils Support Program. The purpose of these additional subtasks is to (1) evaluate whether onsite materials are suitable for constructing landfill liners and caps, (2) identify a potential landfill site at RMA, and (3) evaluate the suitability of the proposed site for hazardous waste disposal.

The scope of this Work Plan includes a description of the Material and Area Feasibility field subtasks. The Material Feasibility subtask includes borrow area selection, test fill construction, and infiltration testing (sealed double-ring infiltrometer [SDRI] testing and two-stage borehole [TSB] permeability testing). The Area Feasibility subtask consists of 3 deep boreholes (continuously cored for geologic data) and up to 30 boreholes (continuously sampled for geotechnical and geological data).

This Work Plan is organized to present the requisite task background and objectives; describe each subtask's design; provide construction, drilling, and sampling procedures for the field programs; present proposed geotechnical testing methods; and discuss data evaluation and reporting procedures and protocols. Section 1.2 of this document presents task background and objectives; Section 2.0

presents test fill construction, testing procedures, and data evaluation; and Section 3.0 presents

geologic and geotechnical subtask design, procedures, geotechnical testing, and data evaluation.

1.2 Task Background and Objectives

The primary objective of this task is to collect soil data to support the Detailed Analysis of Alternatives (DAA) portion of the Onpost Operable Unit at RMA and the Record of Decision (ROD). RMA task background and specific task objectives are discussed in the following subsections.

1.2.1 Task Background

RMA was established in 1942 by the Army as a manufacturing facility for the production of chemical and incendiary munitions. Military, industrial, and agricultural chemicals, primarily pesticides and herbicides, were also manufactured at RMA by several lessees from 1947 to 1982. The industrial waste liquid produced from operations performed by the Army and its lessees was initially discharged to Basin A, an unlined basin in Section 36. Subsequently, liquid wastes were discharged to other unlined basins and, after 1956, to Basin F, which was asphalt-lined. Although solid wastes were disposed of primarily in Section 36, other onpost disposal sites were also used. Some of the basins, pits, burn sites, sewers, and structures (buildings, pipes, and tanks) became sources of soil and groundwater contamination as a result of spills, leaks, or other releases.

Based on the National Contingency Plan (NCP) and Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) guidance, and consistent with the FFA, the Development and Screening of Alternatives (DSA) portion of the FS was performed to establish remedial alternatives capable of achieving the remedial action objectives (RAOs) for RMA. Alternatives retained after the screening process are currently being further evaluated as part of the DAA portion of the FS.

The soils DSA focused on several remedial alternatives for the soils medium including onpost landfilling of materials and capping. The soils medium that is addressed in the soils DSA consists of unsaturated soils, bedrock, fill material, process water lines, chemical and sanitary sewer lines, lake

sediment, and soil/debris mixtures in disposal trenches or landfills; the term "soils" is used for convenience in this document to refer to any of these materials. Recognizing that interactions will occur between soils and other contaminated media such as structures and groundwater during the implementation of remedial alternatives, the impacts and interactions of other media on remedial approaches developed for RMA are being addressed in the DAA portion of the FS. Since landfilling and capping remedial alternatives were retained as remedial alternatives in the DAA, the Program Manager for Rocky Mountain Arsenal (PMRMA) identified additional data needs that are required to support completion of the DAA. The data needs addressed under this task are discussed below.

1.2.2 Task Objectives

The objective of the test fills and infiltration testing of the test fills (material feasibility) is to evaluate whether onsite materials are suitable for constructing landfill liners and caps. The objective of the geologic/geotechnical borings (area feasibility) is to obtain adequate data regarding the geology and geotechnical characteristics of the site to evaluate the feasibility of constructing a landfill in the existing foundation materials. The work will include constructing two clay soil test fills and conducting permeability tests on the test fills to evaluate the suitability of onpost materials for use as liner and cap material. The work also includes coring, geophysical logging, soil sampling, and physical property testing to evaluate the geologic and geotechnical characteristics of the preferred landfill area.

This program will support the evaluation of remedial alternatives in the DAA involving onpost landfilling of materials and capping. The program is scoped as an FS-level investigation to validate and refine feasibility and cost information in the DAA-FS. Investigations will be conducted in a manner consistent with the NCP and CERCLA as amended by the Superfund Amendments and Reauthorization Act of 1986 (SARA).

2.0 TEST FILL CONSTRUCTION AND TESTING

This section presents the test fill construction and testing program to be performed under the FS

Soils Support Program, Subtask 1 entitled, Material Feasibility. The test fill construction and testing

program includes the construction of two test fills using two soil types native to RMA and the

subsequent field testing of the test fills for in situ permeability. The Material Feasibility portion of

the FS Soils Support Program has two main objectives: (1) to verify that onpost soils are capable of

meeting the required liner permeability of less than or equal to 1×10^{-7} centimeters per second (cm/s)

and (2) to evaluate the optimum lift thickness, moisture content, density, and the compactive effort

necessary to achieve this permeability. Permeability of the constructed test fills will be measured

both in the field and in the laboratory to evaluate the suitability of the materials and methodology

used to construct the test fills. Adjustments to the material type, density, and moisture content of

the recompacted soil liner may be necessary pursuant to the results of the test fill program.

2.1 Test Fill Location and Proposed Borrow Material

To simulate future landfill clay liner construction conditions as closely as possible, the test fills will

be constructed at the site currently considered to be the preferred landfill location, the western half

of Section 25 (Figure 2.1) (Ebasco, 1988). The site was approved by the U.S. Fish and Wildlife

Service (USFWS) prior to construction of the test fills.

Figure 2.1 also illustrates the two proposed borrow areas where soil for the test fill construction will

be excavated. The borrow areas were identified in the Draft Final Feasibility Study Soils Support

Program Report (HLA, 1994a) to contain suitable borrow material. The borrow sites were also

approved by USFWS for excavation provided the areas were reseeded following excavation. The

revegetation plan for the borrow areas is presented in Section 2.2.

2.2 Test Fill Construction Procedures

Each of the proposed test fills will be approximately 100 feet long by 40 feet wide at the top of the

test fill (Figure 2.2). The 100-foot length and 40-foot width will allow the SDRI and TSB

permeability testing equipment to be placed at the recommended 12 to 15 feet from the sides and slopes of each test fill to avoid encountering edge effects (as shown in Figure 2.3). The recommended spacing is designed to avoid contact with the sides and slopes of the test fill where the lift construction may not meet minimum specifications and therefore may not be representative of most of the clay liner test fill.

As described below in the construction procedures, the test fills will be constructed by first stripping approximately 4 inches of topsoil, then placing the various test fill layers on the prepared subgrade. Test fill soil will be processed for clod size and moisture content prior to compaction. This will be accomplished in the large processing areas adjacent to the test fills.

Each test fill surface will slope uniformly to one side at a 2 percent slope. This design will provide positive drainage and will simulate actual liner construction conditions. Surface-water run-on control will not be necessary because of the above-grade configuration of the test fills.

Upon completion of the test fills, the area will be regraded to smooth out the contours of the test fills and the immediate surroundings. The stockpiled topsoil will be spread over the test fills and other disturbed areas. This topsoil will prevent desiccation cracking of the test fill surface and also serve as the final "closure" layer of the test fill. No further closure activities, other than seeding and mulching, will be performed.

2.2.1 Excavation

Topsoil will be stripped from the affected area of the borrow sites prior to excavation of actual test fill material. This topsoil will be stored adjacent to each respective borrow site. Necessary volumes of soil from each borrow area will be excavated under the supervision of the Engineer (HLA). The Engineer will visually inspect all borrowed material prior to transport. Scrapers and trucks will be used to excavate and transport borrow soils to the prepared stockpile areas. Once the excavation is

completed, the borrow pits will be graded smooth, covered with the adjacent stored topsoil, and seeded.

2.2.2 Processing Area Preparation

The processing areas adjacent to each proposed test fill will be stripped of topsoil. The surface will be smoothed (proof-rolled) with the pneumatic tires on the front-end loader. This topsoil will be stored adjacent to the processing areas.

2.2.3 Test Fill Area Preparation

The test fill area will be stripped of topsoil. The topsoil will be stored adjacent to the test fills along with topsoil from the processing areas. Once stripping of the surface soil is completed, the test fill areas will be proof-rolled, scarified, and smooth-drum compacted.

2.2.4 Soil Processing

This portion of the work will be completed prior to actual test fill placement to allow adequate time for clod size reduction, moisture addition, blending, curing, and testing to verify the optimal moisture content range has been achieved. These activities will be performed using a dozer to spread and move soil, a disc harrow or other appropriate equipment to reduce clod size and blend moisture into soil, and a water truck with a sprayer to adjust moisture content. Tests to be performed on processed material are included in Table 2.1.

2.2.5 Subgrade Preparation and Working Layer Placement

To ensure that there is no head build-up in the test fills (which could adversely affect the permeability test results), the test fills will be constructed on a foundation of in situ sandy soil. The sandy soil will be thoroughly wetted to a depth of at least 1 foot prior to placement of the next layer to avoid surface tension and capillary effects on downward infiltration. Directly above the wetted sandy soil will be a 6-inch (compacted lift thickness) layer of clay borrow material that will be designated a "working layer." This layer is important to ensure proper placement, bonding, and compaction of the first lift within the test fills.

Explanation

- °013   Boring location for Stage 1 of Boring Program - Nunn Clay
- ▲013   Boring location for Stage 1 of Boring Program - Weld Clay
- ■013   Boring location for Stage 1 of Boring Program - Bresser Clay
- ○013   Boring location for Stage 2 of Boring Program - Nunn Clay
- △013   Boring location for Stage 2 of Boring Program - Weld Clay
- +101   Surface sample location for Stage 2 - Berm Material

┼┼┼┼┼	Railroad
┈┈┈┈	Ditch or stream
── ──	Paved road
═ ═ ═	Unpaved road
16	Section number

Containment Systems

	Physical barrier
	Hydraulic barrier
	Recharge trenches
	Barrier wall

Note:
The boring identification numbers shown have been abbreviated. The full boring identification number contains the soil unit before the number and 93 for 1993 at the end. For example, the complete boring identification number for Boring 012 is WEB01293.

0   1500
Scale in feet

N

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Commerce City, Colorado

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Harding Lawson Associates

Figure 2.1

Site Vicinity Map

2-2A

RMA/91907400 10/18/94 LD2

14   18   17

13   19   20

24   30   29

23   25   32

26   31

35   36

Borrow Area B1
Borrow Area B2
Borrow Area A
Soil Transport Traffic Route
Approximate Location of Onpost Hay Mulch
Approximate Location of Test Fills and Processing Area
North Boundary Containment/ Treatment System
Sewage Treatment Plant
Sanitary Landfill
Storage Area
North Plants
Former Basin A
Former Basin B
Former Basin C
Basin A Neck/ Treatment System
Submerged Quench Incinerator
Liquid Storage Tanks
Pond
North Bog

9th Avenue
8th Avenue
F Street
E Street
D Street

Typical Test Fill Section

- 4" Topsoil
- 6" Compacted Clay Lift (typ.) (6 lifts)
- 6" Working Layer
- SDRI Outer Ring
- 40'
- 3'
- Permeable Native Soil
- Proof-rolled Subgrade
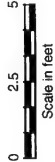
Explanation

SDRI   Sealed double-ring infiltrometer

Typ.   Typical

Note: Maximum slope shall be 2H:1V on long sides and 3H:1V on ends of test fill. Minimum slope shall be 3H:1V on long sides and 4H:1V on ends of test fill.

0   2.5   5
Scale in feet

Figure 2.2
Typical Test Fill Section

2–2B

Typical Cross Section

25 Horizontal
10 Vertical

Scale in feet
0

Soil
Processing Area

Soil
Processing Area

3' Clay Test Fill

Topsoil

6' Working Layer

SDRI

SDRI

Topsoil

6' Working Layer

3' Clay
Test Fill

Slope

Plan View

25

Scale in feet
0

265'

150'

80'

Test Fill #2
Soil Processing
Area

Slope

Access

Slope

Compacting Direction

Slope 2%

Test

Fill

#2

Slope 2%

40'

Borehole

100'

Slope

SDRI

15'-Wide Access Strip

100'

Slope 2%

Test

Fill

#1

SDRI

Slope 2%

40'

Access

Compacting Direction

Slope

Test Fill #1
Soil Processing
Area

Slope

80'

150'

Explanation

SDRI Sealed double-ring infiltrometer

● Two-stage borehole

Native soil

Working layer

Clay test fill

2.2.6   Test Fill Placement

Once the subgrade and working layer are completed for each test fill, and the specified moisture content of the processed soil has been achieved, placement of the test fills will be initiated. Using a front-end loader, dozer, or a backhoe, processed soil will be moved to the test fills. The soil will be spread by a dozer into 8- to 9-inch loose lifts. A heavy tamping foot or sheepsfoot compactor (Caterpillar Model 825 or equal) will be used to compact the clay into 6-inch compacted lifts. Six lifts will be constructed to form the 3-foot-thick test fill. Each lift surface will be scarified and/or rewetted as necessary to ensure adequate bonding between lifts.

Based on the results of the earlier soils testing program, failure to meet a specification of 95 percent of the Standard Proctor maximum dry density (American Society for Testing and Materials [ASTM] D 698-78) will require removal and replacement of that lift.

Moisture content shall be in the range of 1 to 4 percent wet of optimum moisture. Each compacted lift will be tested using a nuclear gauge to verify that the compaction and moisture content specifications have been met. Additionally, to determine the appropriate number of equipment passes to meet the compaction requirement, the density will be measured after every two passes for each of the first three lifts. This methodology will (1) enable the last three lifts to be constructed with the appropriate number of lifts and minimal testing and (2) enable development of a "compaction vs. equipment passes" curve to use in preparing method specifications for the actual clay liner. It is anticipated that three density test locations will be used for each lift at each test fill. These and other tests that will be performed during construction are listed in Table 2.1.

2.2.7   Test Fill Completion and Permeability Test Preparation

To complete the test fills and provide a smooth, uniform surface, the top of the uppermost completed lift will be compacted and sealed with a smooth-drum roller. The completed test fill surface must be protected from excessive desiccation cracking prior to and during the field permeability tests. Therefore, a topsoil layer or plastic sheeting will be placed over both test fills, except for the

Table 2.1: Test Fill Testing Program (Per Test Fill)

Test	Method	Stockpile	Frequency During Construction	Post-Construction
Moisture Content	Oven drying ASTM D2216-90	3 initial; as needed during processing (approx. 12); 3 final; estimate 18 total	3 per lift (18 total)	--
Moisture Content	Nuclear gauge ASTM D3017	--	3 per each 2 passes per lift for first three lifts; 3 per lift for lifts 4, 5, 6; estimate 45 total	--
Atterberg Limits	Grab sample ASTM D4318-84	3	--	--
Grain Size (incl. clay content)	Sieve and hydrometer analysis ASTM D422-63	3	--	--
		3	--	--
Optimum Moisture Content and Max. Dry Density	Standard Proctor test (grab samples) ASTM D698-78	3	--	--
In-place Density (% compaction)	Nuclear gauge ASTM D2922	--	3 per each 2 passes per lift for first three lifts; 3 per lift for lifts 4, 5, 6; estimate 45 total	--
Lift Thickness (loose)	Manual	--	25-foot intervals down centerline of test fill	--
Lift Thickness (compacted)	Manual	--	25-foot intervals down centerline of test fill	--

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Table 2.1 (continued)

Test	Method	Stockpile	Frequency During Construction	Post-Construction
Final Test Fill Thickness	Survey	--	--	3
Lift Bonding	Visual: Test Pits	--	2 per lift (manual)	3 (backhoe)
Laboratory Permeability	Shelby tubes, flexible wall permeameter (falling-head test) ASTM D5084-90	--	--	3
Field Permeability (large-scale)	Sealed double-ring infiltrometer (SDRI)	--	--	1
Field Permeability (small-scale)	Two-stage borehole (Boutwell)	--	--	5
Shear Strength	Consolidated undrained ASTM D4767-88	--	--	2
Shear Strength	Unconsolidated undrained ASTM D2850-87	--	--	2

--   Not performed
ASTM   American Society for Testing and Materials

immediate areas designated for the SDRI tests. Each SDRI area, approximately 12 feet by 12 feet, will be blocked off and sealed with a tarp until the SDRI test set-up is initiated.

2.2.8 Field Permeability Tests

One SDRI and five TSB permeability tests will be performed at each test fill. The SDRI test will be located in the center of each test fill to avoid edge effects. The TSB tests will be placed at least 15 feet from test fill edges. Figure 2.3 illustrates the proposed test locations. Performance of the tests will include strict adherence to the published testing procedures and thorough documentation of all observations and results as described in Sections 2.3 and 2.4.

Protection of the SDRI areas from temperature changes and damage by animals will be accomplished by building a plywood cover built for each SDRI test site. The SDRI area will be staked and marked with tape to prevent access by unauthorized personnel.

2.2.9 Test Fill Closure

Once the field permeability tests are completed and the testing apparatus disassembled, the SDRI areas and the center strip between the test fills will be seeded with native grasses and wildflower seed and mulched with native grass hay as specified by the USFWS.

2.2.10 Test Fill Documentation

In addition to the test results from the formal testing program outlined in Table 2.1, the following information on test fill construction will be documented:

- Field description of the borrow material used in each test fill

- Soil processing procedures

- Soil transport traffic routes

- Dozer spreading and grading patterns

- Lift thicknesses (loose and compacted)

- Compaction patterns and number of passes

- Test locations

- Lift bonding check results and observations

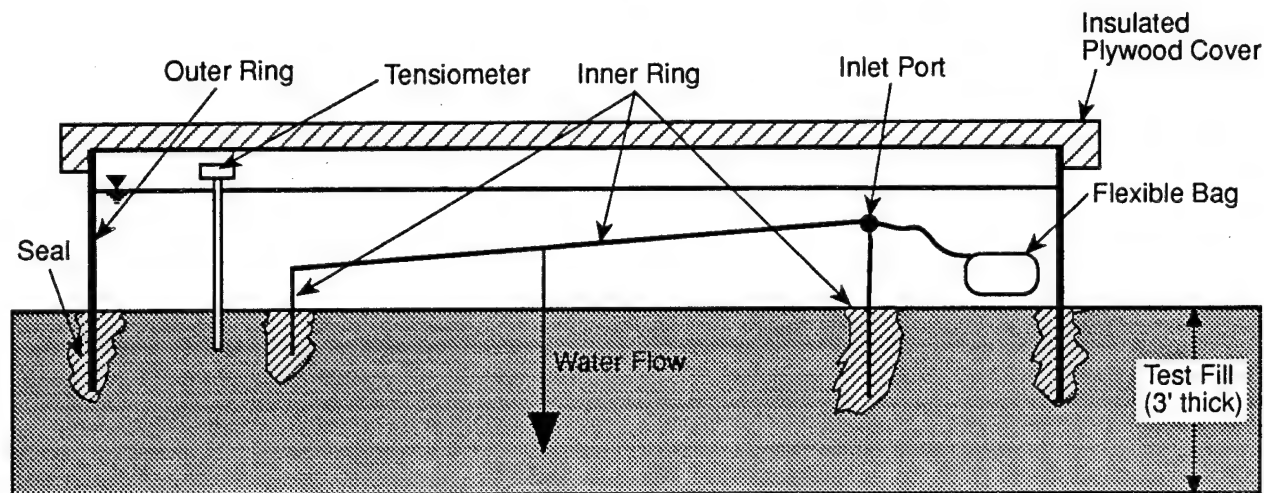- Any failed results and corrective action taken

2.3 Sealed Double-Ring Infiltrometers

·This section describes the test method, the installation, monitoring, and data reduction and evaluation procedures to be used for the SDRI tests. The SDRI test measures the vertical infiltration rate of water through the constructed test fills. The SDRIs are specifically designed to measure low infiltration rates in the range of 1×10^{-5} to 1×10^{-8} cm/s. Based on previous laboratory results (HLA, 1994a), the permeability of the soil at the proposed borrow areas ranges from 3.02×10^{-9} to 8.24×10^{-8} cm/s. This range of soil permeability coincides with the optimal range for the SDRI test method.

A summary of the SDRI test method is provided in Appendix A along with the manufacturer's installation and operating instructions. A synopsis of the test method from the installation instructions (Appendix A) is as follows: The SDRI consists of a 12-foot by 12-foot outer ring and a 5-foot by 5-foot inner ring, as illustrated in Figure 2.4. The rings are grouted within trenches excavated into the top of the test fill. The outer ring is installed at a depth of 14 to 18 inches below ground surface (bgs), and the inner ring is installed to a depth of 4 to 6 inches bgs. After grouting is complete, the area between both rings is filled with water. The outer ring area is filled to a depth of approximately 12 inches, which completely submerges the inner ring. The inner ring area is sealed by placing a cover over the top of the inner ring. This seals the water within the inner ring from the atmosphere.

Monitoring the flow of water in the SDRI is accomplished by filling a flexible bag with a known weight of water and connecting the bag to a port on the inner ring. As the water infiltrates the ground and leaves the sealed inner ring, it is replaced with an equal amount of water drawn in from the flexible bag. After a specified time interval, the flexible bag is removed and weighed. The weight loss is then converted into milliliters of water that has infiltrated into the test fill. Infiltration rate is calculated using an equation with the following parameters: the volume of water loss, the

Cross Section
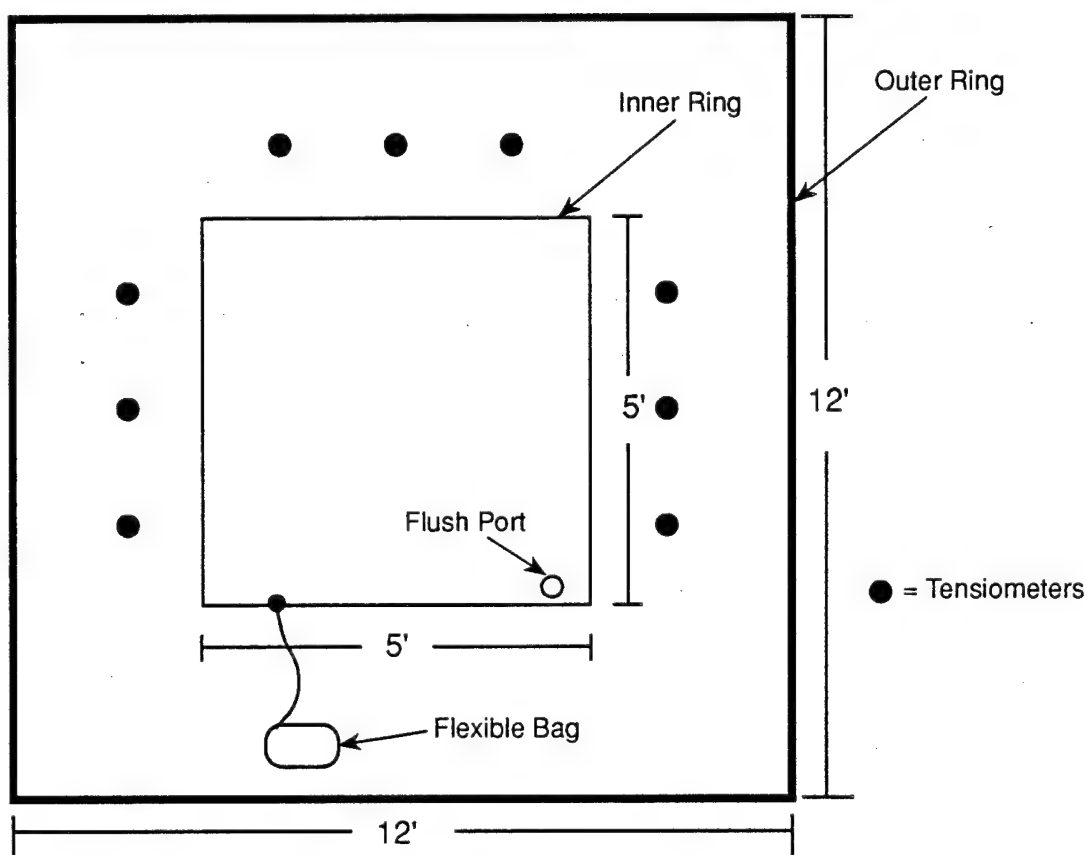
Outer Ring — Tensiometer — Inner Ring — Inlet Port — Insulated Plywood Cover

Flexible Bag

Seal

Water Flow

Test Fill (3' thick)

Not to scale

Plan View

Inner Ring

Outer Ring

5'

12'

Flush Port

5'

12'

● = Tensiometers

Flexible Bag

Not to scale

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    Program Manager for
    Rocky Mountain Arsenal
    Commerce City, Colorado

Prepared by:
    Harding Lawson Associates

Figure 2.4

Proposed Sealed Double-Ring Infiltrometer Construction Schematic

area of the inner ring, and the interval of time that the bag was connected onto the inner ring. The tests will be monitored until the infiltration rate reaches a steady state. Upon completion, a plot of the infiltration rate over time is prepared. For the purposes of this test, the specified permeability value to be achieved is 1×10^{-7} cm/s. It is anticipated that the tests will require monitoring for approximately two to three months.

2.3.1 Sealed Double-Ring Infiltrometer Installation Procedures

The SDRIs will be installed immediately following completion of the test fills. The SDRIs will be installed according to manufacturer's installation instructions under the direction of the installation subcontractor, Mr. Steve Trautwein of Trautwein Soil Testing Equipment (manufacturer of the SDRIs). Further information regarding the installation of the SDRIs is presented in Appendix A.

2.3.2 Sealed Double-Ring Infiltrometer Monitoring Procedures

SDRI monitoring procedures will include flow measurements, water temperature readings, water level measurements, swell measurements, and tensiometer readings. In general, the readings will be taken on a daily basis until the infiltration rate slows sufficiently to allow measurements to be collected every several days. The readings will be taken by field personnel trained and experienced in taking SDRI measurements. The readings will be recorded in field logbooks and then transferred onto the SDRI data forms (Appendix A).

2.3.3 Sealed Double-Ring Infiltrometer Data Reduction and Evaluation

Data reduction and evaluation will be accomplished by transferring the field measurements recorded on the data forms onto computer spreadsheets for ease of computation. The infiltration rate can be determined by using the following equation:

$I = Q/(At)$

where:

I = infiltration in cm/s

Q = volume of flow in cubic cm

A = area of flow in square cm

t = time interval in seconds

There are two factors that can have a significant effect on the infiltration rate of the SDRI: temperature changes and soil swelling. The field measurements of temperature and swell will be reviewed to evaluate whether they could have affected the infiltration rates of the SDRIs. If either factor is found to be significant such that it could account for as much as 10 percent of the total infiltration of the SDRI, corrections for these effects will be made.

The hydraulic conductivity within the SDRI is calculated by using the following equation:

$k = Q/(iAt)$

where:

k = hydraulic conductivity in cm/s

Q = volume of flow in cubic cm

t = time interval in which Q was determined in seconds

i = $\Delta h / \Delta s$ (gradient) dimensionless

Δh = head loss

Δs = length of flow path for which Δh is measured

A = area of flow in square cm

Plots of the infiltration rate and hydraulic conductivity will be evaluated concurrently to evaluate when the tests can be terminated as discussed above (i.e., when steady state conditions or the desired hydraulic conductivity has been achieved).

2.4   Two-Stage Borehole Permeameters

This section presents the installation, monitoring, and data reduction and evaluation procedures for the TSB permeameter tests. The TSB procedure is a falling-head infiltration test conducted in a cased borehole. The first stage of the TSB procedure is performed with a cased borehole that is open at the bottom. The first stage of the test is used to calculate vertical permeability (k_v). The second stage of the test is conducted after advancing the borehole another 6 to 8 inches below the bottom of the casing and measuring the flow rates to calculate horizontal permeability (k_h).

2.4.1   Two-Stage Borehole Permeameter Installation Procedures

Five TSB permeameters will be installed at each test fill immediately following installation of the SDRIs. The TSBs will be installed following the installation procedures described in Appendix B. The boreholes will be excavated using a hand auger and reamed to the desired depth. Polyvinyl chloride (PVC) casing will be grouted in place, and the surface of the borehole will be completed with the measurement stand pipe and fill tube, as illustrated in Figure 2.5.

Along with five TSBs at each test fill, one temperature effect gauge (TEG) will be installed at each test fill. The purpose of the TEG is to measure any changes in flow rates that could be the result of temperature changes during the monitoring period. Field measurements on previous TSBs have indicated that rising temperature causes the water column in the standpipe to expand, thereby causing a lower apparent flow rate (see Appendix B). The net effect is a lower apparent permeability. If the temperature decreases during the monitoring period, then the converse is true. For this reason, TEGs will be installed and monitored throughout the testing program.

TEGs are set up and installed similarly to the typical TSB with one exception; the bottom of the casing is sealed with a cap. Because there is no flow of water from the TEG, any changes in the readings must be the result of changes in the ambient air temperature and/or barometric pressure. As described in Appendix B, any changes noted in the TEGs can then be corrected for in the TSB

measurements. The construction of the SDRI and the TSB will be supervised by the SDRI manufacturer's representative.

2.4.2 Two-Stage Borehole Permeameter Monitoring Procedures

Monitoring of the TSBs will include flow measurements, water temperature readings, and TEG readings. The monitoring equipment manufacturer's operating procedures are presented in Appendix B. The readings will be taken by field personnel trained and experienced in TSB monitoring. The readings will be recorded in field logbooks and then transferred onto the TSB data forms (Appendix B).

2.4.3 Two-Stage Borehole Permeameter Data Reduction and Evaluation Procedures

Data reduction and evaluation will be accomplished by transferring the field measurements recorded on the data forms onto computer spreadsheets for ease of computation. Apparent permeability for both Stage 1 and Stage 2 of the TSB tests will be calculated using the following falling-head test equations as specified in the test method calculations (Appendix B):

$$k = R_T G \ln(H_1/H_2)/t_2 - t_1)$$

where:

k = permeability in cm/s

H_1 = initial head at $t = t_1$

H_2 = initial head at $t = t_2$

t_1 = initial time

t_2 = final time

$\ln$ = natural logarithm

G = geometric constant, depends on the individual test dimensions and is calculated separately for each test

$\dot{R}_T$ = kinematic viscosity correction to water at 68 degrees Fahrenheit as defined in ASTM D5084
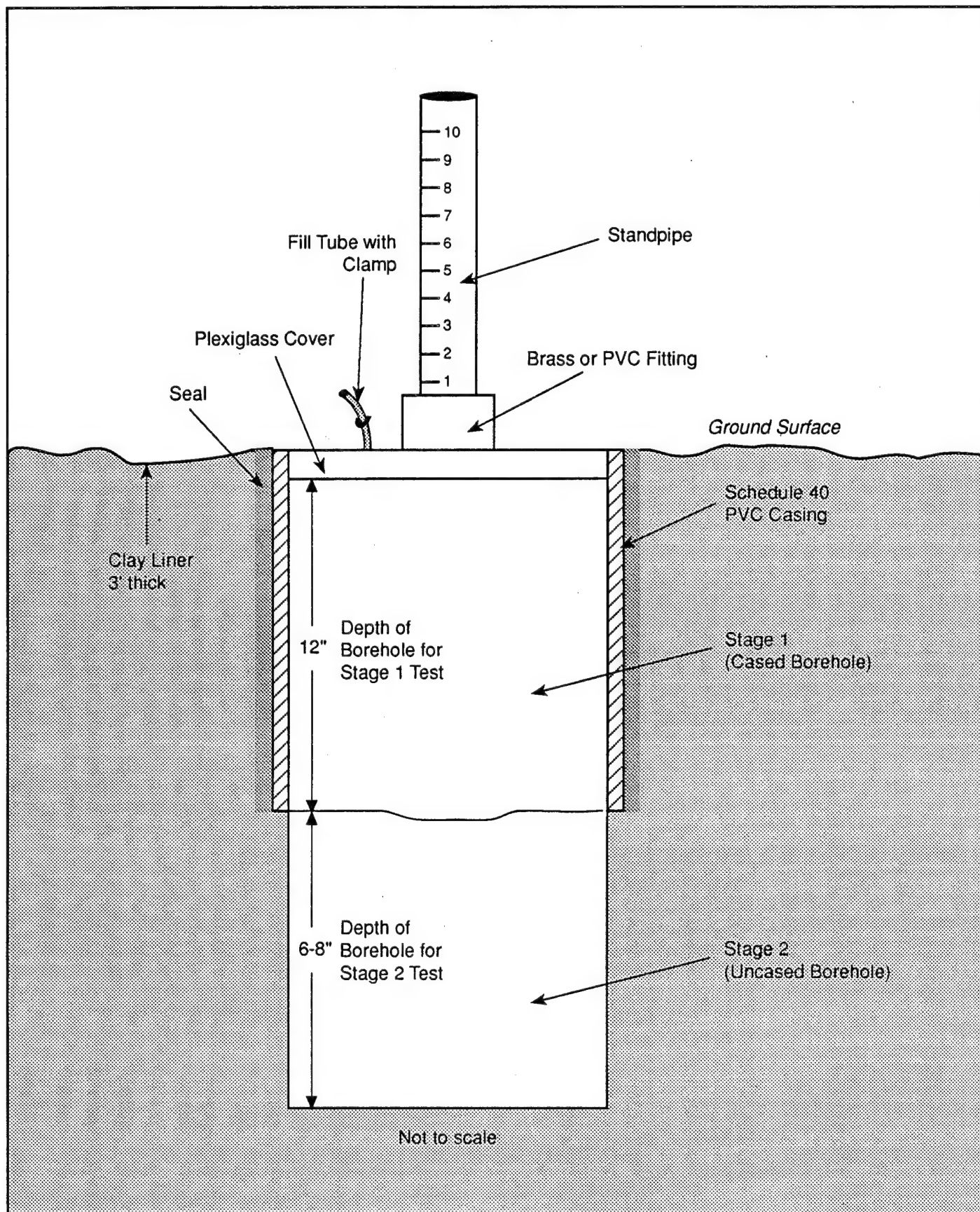
Fill Tube with Clamp

Standpipe

10
9
8
7
6
5
4
3
2
1

Plexiglass Cover

Brass or PVC Fitting

Seal

Ground Surface

Schedule 40 PVC Casing

Clay Liner 3' thick

12"

Depth of Borehole for Stage 1 Test

Stage 1 (Cased Borehole)

6-8"

Depth of Borehole for Stage 2 Test

Stage 2 (Uncased Borehole)

Not to scale

Prepared for:
    Program Manager for
    Rocky Mountain Arsenal
    Commerce City, Colorado
Prepared by:
    Harding Lawson Associates

Figure 2.5

Proposed Two-stage Borehole Permeameter Installation (cross section)

Throughout each stage of the tests, variations in apparent permeability will be evaluated, and the test will be terminated when steady-state conditions are achieved. It is anticipated that each stage of the two-stage test will require monitoring for approximately three weeks.

3.0 GEOLOGICAL AND GEOTECHNICAL DRILLING, SAMPLING, AND TESTING

This section describes the geological and geotechnical drilling, sampling, and testing activities to be conducted under Task 93-03. Section 3.1 describes the selection process for locating the three deep boreholes within the proposed landfill site as well as the procedures for drilling, coring, sampling, and geological and geophysical logging. Section 3.2 describes the rationale for shallow soil boring locations within the proposed landfill site. Drilling, coring, and sampling procedures are also described. Section 3.3 describes the chain-of-custody (COC) procedures. Section 3.4 describes the geotechnical testing, and Section 3.5 describes data evaluation and reporting.

3.1   Borehole Coring Program

This subsection describes the rationale for the selection of the proposed landfill siting deep borehole locations, drilling equipment and procedures, and geological and geophysical logging procedures.

3.1.1   Borehole Locations

Three 150- to 175-foot boreholes will be located in the western half of Section 25 at RMA to further characterize the geology of this area as a potential landfill site. The borehole locations were selected by reviewing existing geologic logs and cross sections prepared from those logs. The boring locations were placed between existing well clusters and known borings to gather new data. These boring locations may be subject to minor changes in the field (with approval of PMRMA). Figure 3.1 shows the proposed borehole coring locations in Section 25.

3.1.2   Drilling, Coring, and Geophysical Logging Equipment and Personnel

A drilling rig and crew will be furnished by the U.S. Army Corps of Engineers Waterways Experiment Station (WES). The drilling rig is capable of both auger and rotary drilling. The WES drilling crew will also provide a supply truck and water truck. Drilling in the alluvium will be accomplished using an 8-inch-outside-diameter (OD) solid auger. Continuous core samples of the alluvium will be collected by hammering a 24-inch-long, split-barrel sampler in the open borehole. Continuous cores

of bedrock will be collected in a 5-foot-long, 4.5-inch-OD split-spoon sampler using a rotary core drilling technique.

Drilling activities will be directed by an HLA field geologist. The geologist will be responsible for directing the drilling crew, logging the core samples recovered, and directing the geophysical logging subcontractor (Colog, Inc.).
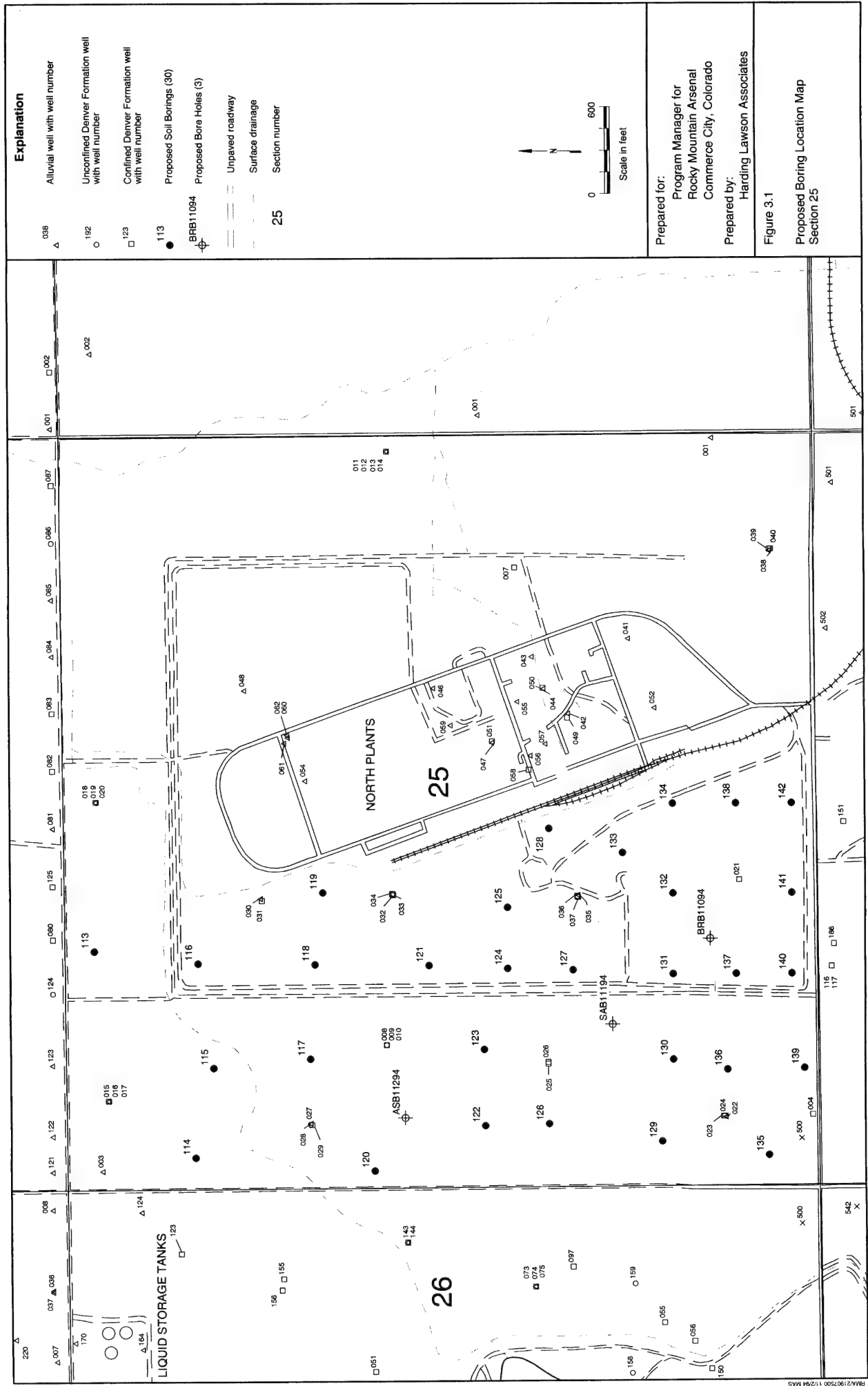
3.1.3 Borehole Coring Procedures

Before field work is started, the geologist will be responsible for assuring that personnel (1) read and sign the accident prevention safety plan, (2) are trained to operate the field equipment, and (3) understand the field procedures described in this Work Plan. All work will be performed in accordance with the Final Accident Prevention Plan (HLA, 1994b).

The boreholes are specifically located outside areas of known contamination; therefore, rigorous decontamination procedures normally followed when soil samples are collected for chemical analysis are not necessary for this program. However, before starting and following completion of drilling operations, the drilling rig and associated downhole equipment will be decontaminated at the CERCLA Wastewater Treatment System (CWTS). Decontamination of downhole equipment between each borehole will only be necessary to maintain proper working order of the equipment.

A lithologic log will be prepared for split-spoon and core samples recovered at each borehole. The samples will be logged immediately in the field and in more detail later using geophysical logs for comparison. These logs will include alluvium and bedrock descriptions, stratigraphic features, details of observed structural features, and other pertinent information. Sample recovery will be noted on the field logs, and the samples scanned with a photoionization detector (PID).

Colog, Inc. (Colog), Golden, Colorado, will provide geophysical logging services during the coring program. Colog will geophysically log each completed borehole after coring has been completed.

Explanation

038 △	Alluvial well with well number
192 ○	Unconfined Denver Formation well with well number
123 □	Confined Denver Formation well with well number
113 ●	Proposed Soil Borings (30)
BRB11094 ⊕	Proposed Bore Holes (3)
— — —	Unpaved roadway
··········	Surface drainage
25	Section number

Scale in feet

0   600

N

Prepared for:
Program Manager for
Rocky Mountain Arsenal
Commerce City, Colorado

Prepared by:
Harding Lawson Associates

Figure 3.1

Proposed Boring Location Map
Section 25

3–2A

NORTH PLANTS

25

26

LIQUID STORAGE TANKS

Colog will use the following borehole geophysical techniques: normal resistivity, spontaneous potential, single-point resistivity, gamma, full waveform sonic, neutron, and caliper. The geophysical logging tools will be calibrated for the methods specified above before logging operations are initiated and after logging operations are completed. The calibrations will be recorded and submitted to HLA for review. The depth indicators for the geophysical logging tools will be set so that zero corresponds with the ground surface before each run. The geophysical logging tools will be run twice over an interval in each borehole. The results of both runs will be compared as a means of verifying the repeatability of results. The geophysical logging equipment will be decontaminated before and after working on this project. Field plots of the geophysical logs will be prepared with an expanded scale of 10 feet per inch. The final logs will be plotted on vellum.

Upon arrival at individual boring sites, the following procedures will be implemented:

1. Water to be used in drilling and grouting will be obtained from a Contracting Officer's Representative (COR)-approved water source.

2. The HLA geologist shall check the boring location stake against the site location map to verify the borehole location and perform borehole clearance for buried utilities and metallic objects.

3. PID background readings will be taken before intrusive activities begin. Field documentation of all boring activities will begin at this time and include the following information: boring number; date; and pertinent observations such as weather, surface conditions, and field equipment identification numbers. It is anticipated that the borings will be drilled in modified Level D personal protective equipment (PPE).

4. Once drilling starts, PID readings will be taken in the breathing zone and at the top of boring. The PID readings are taken for health and safety monitoring and boring location evaluation. If the PID readings are above background levels, the rig geologist will notify the task manager immediately. The level of PPE will be upgraded according to specific guidelines, and the boring will be backfilled and a new boring location identified.

5. The alluvium will be drilled with a solid-stem (8-inch-OD) auger and sampled with 2-foot split-spoon samplers. Blow counts and sample recovery data will be recorded on preprinted boring logs and in bound field logbooks.

6. After the alluvium has been split-spoon sampled and drilled, it will be cased with 5-inch-inside-diameter (ID) PVC casing, and pressure grouted in place.

7. The PVC casing and grout will be left to set up for a minimum of 24 hours.

8. The bedrock will be continuously cored at each borehole location to a consistent reference datum elevation of approximately 5085 feet. Therefore, one borehole will be less than

150-foot total depth and two boreholes will be greater than 150-foot total depth (see Table 3.1).

9. As the soil and rock core are recovered from the boring, they will be logged by the geologist on the lithologic log.

10. The core will be placed in core boxes with the top and bottom of depth labeled for each section of core. The quantity recovered will also be marked on the core box. The outside of the core box shall be labeled with the boring number and core interval. The core will be stored in Building 785 West until the core is no longer needed, at which time the core will be returned to the site and spread on the ground.

11. A metal mud tank will be used to recirculate the drilling mud during coring. Soil cuttings produced during drilling will be spread evenly around the drill site.

12. Upon completion of the drilling and sampling, the boreholes will be geophysically logged by Colog using the following logging tools:

    -   Normal resistivity

    -   Spontaneous potential

    -   Single-point resistivity

    -   Gamma

    -   Full waveform sonic

    -   Neutron

13. After the boreholes have been geophysically logged, they will be grouted to the surface using a tremmie pipe and pumped with cement/bentonite grout.

14. A stake will be placed at the borehole location with the appropriate boring identification clearly marked. The location and ground surface elevation of the borings will be surveyed by a licensed surveyor and the coordinates will be forwarded to RMA's data management subcontractor, D.P. Associates.

3.2　　　Soil Boring Program

Up to 30 shallow boreholes will be drilled and sampled in Section 25 of RMA to gather detailed geologic and geotechnical data for site characterization.

3.2.1　　　Soil Boring Locations

Geologic boring logs and cross sections from borings in Section 25 and northern Section 36 will be reviewed. The data review and cross sections will help locate the boreholes for the drilling program.

Table 3.1: Projected Borehole Drill Depths

Boring Number	Projected Thickness of Alluvium (feet)	Projected Total Depth (feet)	Projected Surface Elevation (feet)
BRB11094	22.5	175	5260
SAB11194	15.8	170	5254
ASB11294	27.2	145	5232

Borings will be terminated at approximately 5085 feet elevation above mean sea level.

The borehole locations will be cleared for utilities and other subsurface objects using utility maps and a metal detector before any drilling begins.

Boring locations will be selected to optimize the density coverage within the proposed landfill siting area. The most desirable geologic areas for siting a landfill in Section 25 (based on the suitability criteria) will be targeted for study. Figure 3.1 shows the proposed soil boring and coring locations.

3.2.2 Drilling and Sampling Personnel, Equipment, and Procedures

This subsection presents soil boring and geotechnical soil sampling procedures. Procedures for completing COC forms and shipping forms are also discussed.

Field Personnel

Soil boring activities will be accomplished by a two-person field team. The field team will consist of an HLA geologist and an engineering technician (ET). The rig geologist will log each boring in the field as it is drilled, and continuously collect soil samples. The task manager (TM) will review the final field lithologic logs. The ET will assist the rig geologist with sample collection and be responsible for sample handling, packaging, and shipment.

Field Equipment

Borehole drilling will be accomplished using 3-1/4-inch-ID hollow-stem augers and a Mobile drilling rig. Split-spoon samplers and shelby tubes (when necessary) will be used to collect soil samples during drilling. Layne Environmental Services, Inc., will provide a drilling rig, drill crew, water truck, and a vessel to mix grout in for borehole abandonment. HLA will have a truck to transport sample coolers, sample bottles, and sample equipment.

Soil Sampling Procedures

The rig geologist will be responsible for assuring that personnel (1) have read and signed the Site Safety and Health Plan, (2) are trained to operate the field equipment, and (3) understand the field procedures described in this Work Plan.

Each boring location must be cleared for utilities and metallic objects before drilling can begin. The TM will make sure that COC forms and labels are prepared for each borehole. Information from previous boreholes in the areas being drilled will be provided to the rig geologist. Data from the boreholes will be recorded on preprinted boring logs and in bound field logbooks.

Up to 30 borings will be drilled and sampled as part of the area feasibility subtask. The borings will be continuously sampled to a depth of approximately 50 feet or into bedrock. The alluvium will be logged for lithology and blow counts will be recorded to collect geotechnical information. Samples will be collected for geotechnical testing from every 5-foot interval. Undisturbed soil samples will be collected using split-spoon samplers and shelby tubes when necessary. It is anticipated that the borings will be drilled in Level D PPE. The borings have been located outside areas of known soil contamination, and if any sustained readings above background are detected in the field using a PID, drilling will be stopped and the level of PPE will be upgraded as necessary.

Before starting and after completion of drilling operations, the drilling rig and associated downhole equipment will be decontaminated at the CWTS. However, decontamination of downhole equipment between each borehole will not be necessary.

Upon arrival at individual boring sites, the following procedures will be implemented:

1.	Water to be used in drilling and grouting will be obtained from a COR-approved water source.

2.	The HLA geologist or technician shall check the boring location stake against site location maps and pertinent borehole clearance information to verify the boring location.

3.	An HNu PID will be used to obtain background readings before intrusive activities begin. Field documentation of all boring activities will begin at this time and include the following information: boring number; date; and pertinent observations such as weather, surface conditions, and field equipment identification numbers.

4.	Once drilling commences, PID readings will be taken in the breathing zone and at the top of boring. The PID readings are taken for two purposes: health and safety monitoring and boring location evaluation. If the PID readings are above background, the rig geologist will notify the TM immediately, and two actions will be taken. The level of PPE will be upgraded

according to specific guidelines, and the boring will be backfilled and a new boring location identified.

5. Soil cuttings produced during drilling will be spread evenly around the drill site.

6. As the soil core is recovered from the boring, it will be logged by the site geologist on the lithologic log. Soil cuttings from the auger will also be monitored for lithologic changes and noted on the boring log.

7. The samples will be continuously collected and those samples to be analyzed will be selected following completion of the borings and a review of the boring logs.

8. Most samples will be collected in wide-mouth jars for geotechnical analysis. Additional material will be collected in shelby tubes and 5-gallon buckets. The 5-gallon bucket samples will be collected from the auger flights. Sampling technique, sample depth, and fractions collected will be recorded on the lithologic logs, COC forms, and sample tags.

9. The residual soil core will be packaged for storage at Building 785 West until no longer needed, at which time the residual soil cores will be returned to the site and spread on the ground.

10. The borings will be terminated at bedrock or at 50 feet, whichever is encountered first. If groundwater is encountered, the boring will be terminated at the groundwater level.

11. Upon completion of the soil sampling, the boring will be backfilled with portland type I/II cement with 5 percent bentonite. Residual soil will be removed from the auger bits and soil sampling equipment.

12. A stake will be placed at the borehole location with the appropriate boring identification clearly marked. The location of the boring will be surveyed by a licensed surveyor and the coordinates will be forwarded to D.P. Associates.

13. Samples will be labeled and stored onsite pending selection of samples for subsequent geotechnical analyses. At the conclusion of the program, unused samples will be spread evenly over ground surface at a location approved by PMRMA.

3.3   Chain of Custody

The ET will place the correct COC forms within the designated sample cooler before relinquishing the cooler to the rig geologist. These forms include an inventory of the samples and a listing of those persons with access to the samples. The forms will be transported with the samples at all times. Possession of the samples will begin with the sample collectors. All subsequent sample transfers will require the relinquisher and the receiver to sign, date, and record the time of transfer on the COC forms.

Data on final COC forms will be checked by the ET and will include the sample number, sampler's signature, collection date and time, fractions collected, and sample depth. The ET will check these data against the boring logs and field logbooks transmitted by the rig geologist.

At the end of each day, all samples will be brought back to the trailer for packaging. The ET will complete the COC forms and review field logbooks and field data sheets for errors and omissions.

Sample fractions will be repackaged with a layer of plastic bubble wrap below and above the samples, in heavy-duty coolers to ensure that the samples will not break during shipment. COC forms will be placed in waterproof bags in their corresponding coolers. All coolers will be sealed and wrapped in accordance with PMRMA shipping requirements. Evidence tape will be placed across each cooler to ensure that the contents are not violated during shipping. The last person to sign the COC form for each cooler will sign and date the evidence tape. The COC forms will be signed over to PMRMA's transport courier. PMRMA will ship the samples by air freight (Federal Express) to HLA's geotechnical laboratory.

3.4   Geotechnical Testing

A series of laboratory geotechnical tests will be performed on soil samples collected during the soil boring program to evaluate the geotechnical characteristics of the soil in the preferred landfill area. The geotechnical testing will be performed by HLA's geotechnical laboratory in Houston, Texas.

Samples for geotechnical testing will be selected at 5-foot intervals from the boreholes. The following geotechnical tests will be performed:

- Geotechnical testing to include approximately 350 samples for the following tests:
  - Grain-size analysis (ASTM D422)
  - Atterberg limits (ASTM D4318-84)
  - Natural water content (ASTM 2216)

- Remolded compaction (ASTM D698)

- Remolded permeability (EM1110-2-19096) at 90 and 95 percent relative compaction

- Shrink swell (ASTM D427)

- Organic content (ASTM D2976)

- Flexible wall permeameter (ASTM D5084-90)

- Shear strength consolidated undrained (ASTM D4767-88)

- Shear strength unconsolidated undrained (ASTM D2850-87)

Approximately 100 percent of the samples will be analyzed for grain size, Atterberg limits, and natural water content. The remaining tests will be performed on approximately 10 percent of the samples.

3.5   Data Evaluation and Reporting

The data from the borehole coring and geophysical logging, soil boring and sampling, and geotechnical laboratory testing subtasks will be included in the Site Feasibility Report. These data will be integrated into the area feasibility subtask and used to evaluate the feasibility of constructing a landfill using the existing foundation materials.

The borehole coring and geophysical logging data will be used with existing geologic data to generate additional detailed geologic cross sections within Section 25. Drafted lithologic logs of each borehole will be produced. The geophysical logging data will be used to correlate between the boreholes and to compare with the geologic core. A map will be generated that includes the new and existing boreholes, and also shows the location of newly constructed cross sections.

The soil boring geologic logs and geotechnical testing data will be used to generate shallower geologic cross sections and characterize the geotechnical properties of the soil in the western half of Section 25. The geotechnical test results will be submitted to PMRMA through the data management subcontractor, D.P. Associates.

4.0 ACRONYMS AND ABBREVIATIONS

Army   U.S. Department of the Army

ASTM   American Society for Testing and Materials

bgs   Below ground surface

CERCLA   Comprehensive Environmental Response, Compensation, and Liability Act

cm/s   Centimeters per second

COC   Chain of custody

Colog   Colog, Inc.

COR   Contracting Officer's Representative

CWTS   CERCLA Wastewater Treatment System

DAA   Detailed Analysis of Alternatives

DSA   Development and Screening of Alternatives

ET   Engineering technician

FFA   Federal Facilities Agreement

FS   Feasibility study

HLA   Harding Lawson Associates

ID   Inside diameter

K_v Vertical permeability

K_h Horizontal permeability

NCP   National Contingency Plan

OD   Outside diameter

PID   Photoionization detector

PMRMA   Program Manager for Rocky Mountain Arsenal

PPE   Personal protective equipment

RAOs   Remedial action objectives

RMA   Rocky Mountain Arsenal

ROD   Record of Decision

SARA   Superfund Amendments and Reauthorization Act of 1986

SDRI   Sealed double-ring infiltrometer

TEG   Temperature effect gauge

TM   Task manager

TSB   Two-stage borehole

USFWS   U.S. Fish and Wildlife Service

WES   Waterways Experiment Station

5.0 REFERENCES

EBASCO Services, Inc. 1994. *Final report, hazardous waste land disposal facility assessment,* September.

Harding Lawson Associates. 1994a. *Draft final feasibility study soils support program report,* June.

Harding Lawson Associates. 1994b. *Final accident prevention plan (site safety and health plan) for the feasibility study soils support program,* as amended June 1994.

Appendix A

INSTALLATION AND OPERATING INSTRUCTIONS FOR
THE SEALED DOUBLE-RING INFILTROMETER

INSTALLATION AND OPERATING INSTRUCTIONS FOR THE
SEALED-DOUBLE RING INFILTROMETER

1.0 SUMMARY OF TEST METHOD

The Sealed-Double Ring Infiltrometer (SDRI) can be used to measure the vertical, one-dimensional infiltration rate of water through soil. This device provides a means to measure low infiltration rates, those associated with fine-grained, clayey soils, and are in the range of 1×10^{-5} cm/sec to 1×10^{-8} cm/sec. The SDRI is particularly useful for measuring liquid flow through soil moisture barriers such as compacted clay liner or covers used at waste disposal facilities, and amended soil liners such as those used for retention ponds or storage tanks.

A schematic of a typical test installation is shown in Fig. 1. The infiltrometer consists of an outer ring (12' x 12') and an inner ring (5' x 5'). The rings are grouted in trenches excavated in the test area. The outer ring is placed at a depth of 14 in. to 18 in., the inner ring to a depth of 4 in. to 6 in. Both rings are filled with water. The outer ring is filled to a depth of approximately 12 in., submerging the inner ring. The top on the inner ring seals the water within it from the atmosphere.

Installation requires a level, obstruction free work zone approximately 24' x 24'. A primary concern before and during installation of the infiltrometer is desiccation. Before installation, desiccation can be minimized by covering the test area with plastic. Plastic sheets for this purpose, 20' x 100', are readily available at most building supply stores. Spreading a thin layer of soil over the plastic will prevent it from blowing away. During installation, desiccation can be minimized by removing plastic only in areas being worked, and recovering areas once work has been completed. Water should be sprayed on any soil that is exposed for long periods of time.

Measurement of flow is made by connecting a flexible bag, filled with a known weight of water, to a port on the inner ring. As water infiltrates the ground and leaves the sealed inner ring, it is replaced with an equal amount of water drawn in from the flexible bag. After a known interval of time, the flexible bag is removed and weighed. The weight loss, converted to a volume, is equal to the amount of water that has infiltrated the ground. An infiltration rate, usually expressed in cm/sec, is then determined using this volume of water, the area of the inner ring, and the interval of time that the bag was connected to the inner ring. This process is repeated and a plot of infiltration versus time is constructed. The test is continued until the infiltration rate becomes steady or until it becomes equals to or less than a specified value.

The advantage of the SDRI over other infiltrometers is the capability to measure low infiltration rates. This is accomplished by measuring the actual quantity of flow rather than a drop of elevation in the water level and by eliminating evaporation from the ring where measurements are made.

2.0 PARTS LIST

First check to see that the following parts were included with the Sealed Double Ring Infiltrometer :

    a. 4 - aluminum panels approximately 12' x 36"
    b. 1- fiberglass inner ring approximately 5' x 5'

c. 36 - of each of the following: 3/8" round head bolts and nuts
d. 2 - flexible bags with pinch valves and barbed connectors
e. 1 - 1/2" NPT plug fitting
f. 1 - 1/2" NPT fitting with straight-barbed connector
g. 1 - 1/2" NPT fitting with elbow -barbed connector
h. 1 - 15' length of 3/8"od x 1/4"id clear plastic tubing
i. 1 - 1/4" brass plug fitting for sealing end of plastic tubing
j. 1 - tee fitting
k. 4 - rubber gasket strips
l. 1 - tensiometer set (optional -.see Appendix A)
m. 1.-. swell gage (optional - see Appendix B)

In addition to the list above, the following items will be needed to assemble and install the SDRI:

a. 1 - flat bladed screw driver for assembling the outer ring
b. 1 - 9/16" wrench for assembling the outer ring
c. 1 - brick or mason's hammer for excavating trench for the inner ring
d. 1 - adjustable wrench for installing fittings on inner ring
e. 1 - knife or scissors for cutting tubing
f. 1 - trenching machine for excavating outer ring trench
g. 5 - 5 gal. buckets to mix grout and place on inner ring
h. water supply - approximately 1200 gallons is needed to fill rings
i. bentonite grout to place in trenches
j. cover for rings (see Appendix C)
k. 1 - thermometer to monitor temperature in outer ring
l. surveyor's level and rod
m. 1 - scale to measure the depth of the water in the outer ring.
n. grout mixer, shovels, and wheelbarrows for preparing grout.
o. cinder blocks to stand on when connecting fittings to inner ring and also to hold down platform for flexible bag.
p. trowel
q. scale - 4000g capacity sensitive to 1g
r. 1 - 14' 2x4 to use as guide when excavating trench
s. plywood (2'x3'x1/2") to use as splashboard and platform for flexible bags

3.0 ASSEMBLY OF OUTER RING

Installation starts with the assembly of the outer ring as follows:

1. Carefully uncrate the aluminum panels. Save the crate for future shipping.

2. Carefully tilt up two panels and align edges and bolts holes (Fig. 2). Support panels on both ends as they bend easily, particularly in windy conditions.

3. Wipe panel edges clean around bolts holes. Also wipe rubber gasket strips clean.

4. Place a rubber gasket between the panel edges and insert bolts through the holes. Turn nuts on bolts until finger tight.

5. Tilt up remaining panels, one at a time, and bolt edges together as described above.

6. Position outer ring so that it is square. Turn nuts on bolts until snug. Do not overtighten as this will cause panel edges to bow apart between bolts.

4.0 EXCAVATION OF TRENCHES

The area to be tested should be level with a slope no greater than 4" over 12'. Slopes of this magnitude are difficult to detect by eye, so a surveyor's level should be used to check elevations. High and low areas should be noted so that trench depthin these zones can be adjusted to keep rings level.

Procedures for excavating trenches are described below.

4.1 OUTER RING

1. Set the outer ring on the area to be tested so that it is square.

2. Scribe a mark on the ground along the lower edge of the ring.

3. Lift ring and place it aside while trench is being excavated.

4. Place the edge of a 2x4 along scribe marks and paint a line on the ground. Extend line approximately 5' beyond each corner. This will help in aligning trenching machine.

5. Use a trenching machine to excavate the trench (Ditch Witch Model No. 1010 or equivalent). Select a machine that makes as narrow a trench as possible, no more than 4" - 6" in width. The narrower the trench, the less grout needed to fill it. Align the trenching machine with the boom over the paint line. The side of the trencher which removes the excavated dirt should be pointed away from the test area. Offset a 2x4 so that it is parallel to the paint line and against the wheel of the trenching machine inside the test area. Paint a line along edge of 2x4 to mark its position. Use the 2x4 as a guide by keeping the wheel of the trencher against it while excavating the trench. Excavate so that the deepest point is 18". Limit the amount of over excavation at corners as this will only increase the volume of grout needed.

   In some soils, several passes may be needed to reach the required depth. Typically, the higher the plasticity index, the greater the

number of passes. Loose soil in bottom of trench and packed in corners should be removed by hand.

6. Once trench has been excavated and loose material removed, carefully place outer ring in trench to check fit. Make adjustments as needed. The ring should be level ($\pm$1"). After adjustments are made, lift ring out of trench and set aside. Cover trench to keep soil from drying while grout is prepared.

4.2 INNER RING

1. Center fiberglass inner ring within outer ring. Scribe a mark on ground along lower edge of inner ring.

2. Note the orientation of inner ring and set it aside. Also lift outer ring and place it aside. Cover outer ring trench to keep from drying.

3. Use a brick hammer (mason's hammer) to excavate a narrow trench. Trench should be approximately 2" wide and 6" deep. When using brick hammer, it is best to start by digging down several inches in one spot and advancing trench forward by chopping down on soil. Try not to pry soil up as this tends to lift up large wedges, open cracks, and causes the trench to be oversized .

4. Place the inner ring in the trench to check the fit. Excavate any areas where the ring does not fit. Use a surveyor's level to check elevation of the corners of the ring. The inner ring should be level or tilted so that lower end is slightly below horizontal. If lower end of ring is above horizontal, air may be trapped in ring when ring is filled with water.

5. Set the ring aside and cover the trenches.

5.0 INSTALLATION OF THE RINGS

5.1 PREPARATION OF GROUT

A product sold by American Colloid called "Volclay Grout" works well for sealing the rings. Between 15 to 20 bags of Volclay Grout are needed for 4" wide by 18" deep trench. If this product is used, add between 15-20 gallons of water per 50 pound bag. The most convenient way of mixing the grout is to use a 4 bag grout mixer. Two bags of grout can be prepared at a time. First add 15 gallons of water to the mixer and then slowly add the grout . Adding the grout too quickly will result in a mixture with large clumps. Add 15 more gallons of water and then add the second bag. Add additional water as needed until the grout flows easily.

5.2 OUTER RING

1. Prepare enough grout to fill the outer ring.

2. Remove the cover from the outer ring trenches and clean all loose dirt out of the trench.

3. Use a wheelbarrow to place grout in the outer ring trench. Use a sheet of plywood from the outer ring crate as a splash board to guide grout into trench and from getting on ground inside the trenches

4. With one person at each corner of the outer ring, lift it and center it over the trench. Slowly push the ring in place while keeping it level. Once in place, use a trowel to push the grout against both the inside and the outside of the ring , particularly at the corners, to obtain a good seal.

5. Pile loose soil (12" high) all around the outside edge of the outer ring (Fig. 3). This berm will prevent the ring from bowing and will keep grout from being pushed out of the trench when the ring is filled with water. The berm also serves to insulate the outer ring which minimizes temperature changes in the water.

5.3 INNER RING

1. Prepare a thicker mix of grout for the inner ring trench.

2. Remove the cover from the inner ring trench and clean all the loose dirt out of the trench. Also clean off the surface of the area surrounded by the inner ring trench.

3. Fill the trench to within 3/4" of the top. Rod the grout to remove any air pockets.

4. Lift the inner ring and center it over the trench. Lower it into the trench and push it down into place. Use a surveyor's level to check the elevation of the corners of the ring. Make sure that the lower end of the ring is not tilted or raised above horizontal as discussed before.

5. Use a trowel to press the grout against the outside wall of the ring in order to obtain a good seal.

6. Cover the grout to prevent desiccation.

6.0 TENSIOMETERS

If it is desired to know the position of the wetting front during infiltration, it is recommended that tensiometers be used. A description of tensiometers and installation procedures are given in Appendix A.

7.0 SWELL GAGES

If the soil has a high potential for swelling, it is recommended that swell be measured during the test. Swell can be measured by monitoring the vertical movement of the inner ring during the test. As the soil beneath the inner ring swells, it will lift the inner ring. A procedure for monitoring swell is described in Appendix B

8.0 FILLING THE RINGS

It is best to fill the rings slowly so that the seal can be checked for leaks. It is much easier to repair a leak when the water level is low than when it is high.

When filling the inner ring, it is important to realize that water causes an uplift force to act on the ring. If the ring is filled to too high a level, the uplift forces can lift the ring out of the ground. For this reason buckets of water are placed on the inner ring before water is is added to it.

The general procedure for filling the rings is as follows. First, the inner ring is partially filled and let to sit to check its seal. Next, the outer ring filled. The ports on the inner ring are left open so it will fill as the water level in the outer ring rises.

The fittings are attached to the inner ring after the outer ring is filled. The cinder blocks are used to provide a place to stand when attaching the fittings. Place several cinder blocks on the ground in the vicinity of ports on the inner ring. Also place several cinder blocks on the ground just inside the outer ring to provide a place to lay the flexible bag during the test.

Detailed instructions for filling the rings are given below.

8.1 INNER RING

1. Fill two buckets with water and place one on each corners of the low edge of the inner ring. Make sure that the buckets are placed on the edge of the ring and not in the center as this may cause the fiberglass to crack. Try not to spill any water around the inner ring as this will make it difficult to check for leaks around the seal later on.

2. Invert one bucket on the ground near the ports on the inner ring. Fill a second bucket with water and place it on the inverted bucket.

3. Cut a length of the flexible tubing long enough to reach from the bucket to the top port. Use this tube to siphon the water from the bucket to the inner ring. Siphon a total of three buckets (15 gallons) of water into the inner ring.

4. Let the water in the inner ring stand for at least 30 min. Check for leaks in the inner ring seal and repair any that are found.

8.2 OUTER RING

1. Place a piece of plywood from the outer ring crate on the ground between the inner and outer ring. Place a bucket on the plywood. Put the end of the hose that is to used to fill the rings into the bucket.

2. Slowly fill the rings at a rate that will not scour the soil.

3. Should a leak occur, repair it by pushing down on the grout on the inside edge of the outer ring first, then pressing down on the grout along the outer edge.

5. When the water level is at the top port on the inner ring, stop filling and allow the water level in the inner ring equilibrate with the water in the outer ring.

6. Continue to fill the outer ring until the water level is approximately four inches above the top port on the inner ring (a depth of approximately 12"). Use a board or shovel handle to gently tap the inner ring to dislodge air bubbles that are trapped inside. Continue tapping on the inner ring until bubbles cease to emerge from the top port.

7. Remove the buckets from the top of the inner ring.

9.0 INSTALLATION OF FITTINGS

Before installing any fittings into the ports of the inner ring, check that all the threads are wrapped with teflon tape. Screw fittings in slowly at first and check that they are not cross-threaded. The threads in the fiberglass can be stripped easily. Also, do not overtighten the fittings as this may crack the fiberglass.

Detailed instructions for installing the fittings are given below.

1. Find the plug fitting and install it in one of the lower ports.

2. Find the two fittings with the barbed hose connectors. The straight fitting goes in the lower port and the elbow fitting goes into the top port. Saturate the fittings before connecting them to the inner ring.

3. Cut two lengths of tubing, one 3' long and the other 7' long

4. Place the two pieces of tubing under water to saturate them. Be sure that all the air is removed from the tubing before connecting it to the inner ring. Any air remaining in the tube will be drawn into the inner ring.

5. Push one end of the long piece of tubing onto the top port fitting. Find the small brass plug fitting and insert it into the other end of

the tubing. This tube is the flush tube and is used to purge air that has become trapped in the inner ring.

6. Connect the short piece of tubing to the lower port fitting. This is the inlet tube through which flow measurements are made. Fix the open end of the tube to one of the cinder blocks near the wall of the outer ring. Be sure the end of the tube does not float to the surface and suck in air or fall to the bottom and suck in mud.

10.0 COVERING THE RINGS

The rings should be covered throughout the test. Its best to construct a cover of plywood, 2x4's, and insulation. The primary purpose of the cover is to minimize temperature change of the water in the rings, block sunlight, and prevent from wind blowing over the water surface. Supplies and material for building a cover are given in Appendix C.

11.0 DATA COLLECTION

The data collected during the test includes flow measurements, water temperature, water level measurements, swell measurements, and tensiometer readings. Sample data sheets are attached to these instructions. The procedures used to collect these measurements are discussed below.

11.1 FLOW MEASUREMENTS

Measurement of flow during an SDRI test is made using a flexible bag. The bag is filled water, weighed, connected to a port on the inner ring, and submerged in the water of the outer ring. Any water that flows out of the inner ring into the ground will be replaced by an equal amount of water from the bag. Periodically, the bag is removed and weighed to determine the amount of water that was lost.

Besides convenience and simplicity, a key feature of using a flexible bag to measure flow is that a constant pressure difference is maintained across the wall of the inner ring. Consequently, the inner ring does not expand or contract when the water level changes in the outer ring.

The flow measurement data is used to construct a plot of infiltration versus time. For unsaturated soils such as compacted clay liners and covers, infiltration decreases with time at first, changing rapidly at the beginning of the test, and then eventually becoming constant with time as the soil becomes saturated. Consequently, more frequent readings are needed at the beginning of the test and less frequent readings are need as the flow rate becomes steady.

Typically, flow rates at the beginning of the test (< three weeks) range from 1000 ml to 3000 ml per day. One reading per day has been found to be sufficient during this time. When infiltration starts to level out (three to four weeks) one reading in several days is all that is necessary.

Temperature changes of the water in the inner ring can introduce significant error in the flow measurements. A 1 change in water temperature can result in a flow of 50 ml due to volume change of the water in the inner ring as well as in the inner ring itself. To avoid this problem, the bag should be disconnected from the inner ring when the water temperature is within ± 1 °C of the water temperature when the bag was connected. This is particularly important if the flow rate is less than 500 ml/day.

Experience has shown that if the rings are covered and a layer a polystyrene is used for insulation under the cover, the temperature of the water from one morning to the next does not vary by more than 1°C. More consistent readings are obtained if they are taken on a 24 hour basis and the bag is connected and disconnected between 7am and 9am. It should be noted that the water temperature may change by several degrees during the day but that these cyclic variations are only a problem if readings are made at different temperatures. Allowing the bag to remain connected until at least 1500 ml of flow has occurred also helps to minimize the effect of temperature changes on the measurement of infiltration rate.

The bag should never be allowed to empty when connected to the inner ring. When the bag empties, a suction will develop in the inner ring and it may jepoardize the seal. The most likely time that the bag will empty is at the beginning of the test when flow rates are not known. For this reason, the bag should be checked often when first connected. An initial reading should be made after several hours so that a flow rate can be calculated and an estimate of when the bag will empty can be made.

It should be noted that it is not necessary to have the bag connected to the inner ring continuously. If the flow rates are high, (>3000 ml/day) it may be more convenient to connect the bag up to the inner ring for several hours a day and let the inlet tube open in the outer ring for the remainder of the time. Whether the inlet tube is connected to the bag or open to the outer ring does not affect the infiltration rate. Just be sure if the tube is left open that it is propped in such a way that it does no suck in air or soil. If it is desired to measure flows greater than 3000 ml/day, a tee fitting has been provided so that two bags can be connected to the inlet tube at once.

Detailed instructions for using the bag given below.

11.1.1 Filling the bag.

1. Fill a bucket or a 5 gallon water jug with water and allow to stand for 24 hours to degas.

2. Cut a piece of flexible tubing long enough to reach from the bottom of the jug or bucket to a flexible bag laying next to it.

3. Connect the tube to the valve on the bag and siphon water from the jug into the bag until it is filled.

4. Lift the bag above the water surface in the jug. Hold the bag with the inlet port at the top and squeeze it to remove all the air. Squeeze the bag long enough to force the air out of the tube and then lower the bag so that water will flow back into it. Repeat this process until all the air is removed.

5. Once all the air is removed, fill the bag slightly less than full and shut the valve. Avoid completely filling the bag so that the water in it is under pressure.

6. Dry the bag and valve thoroughly. If small amounts of flow are expected (20 ml or less) then be sure that the tube connector remains full of water.

7. Weigh and record the initial weight of the bag to the nearest gram.

11.1.2 Connecting the bag to the inner ring.

1. Connect the bag to the inlet tube as follows. Lower the bag into the water of the outer ring. Orient the valve so that the tube connector is pointed up. Flick the tube connector so that any entrapped air bubbles will be removed from it. It is important that no air bubbles are present in the tube connector or bag as they will be drawn into the inner ring or may even block the flow of water from the bag to the inner ring. With the bag completely submerged push the tube connector into the inlet tube. Lay the bag flat on the cinder blocks. Be sure to position the valve so that it is not folded back onto the bag and possibly pinching off the flow path.

2. Start flow measurements immediately as follows. Use the attached data sheet and record the date and time next to the initial weight of the bag. Carefully open the valve and allow flow to occur.

3. Periodically determine the amount of flow that has occurred as follows. Carefully close the valve and disconnect it from the inlet tube. Be sure to close the valve before handling the bag. Also, be sure to prop up the open end of the inlet tube for the reasons mentioned previously. Record the date and time that the valve was closed . Remove the bag from the inner ring, dry it thoroughly, and record its weight. As before, make sure that the tube connector is filled with water to be consistent. Subtract the final weight from the initial weight to obtain the amount of flow that has occurred.

4. Refill and reweigh the bag if necessary and connect it to the inner ring. Always check to see that the valve on the bag or the inlet tubing has not become clogged. With time, algae may grow in the tube. If this is the case then the tube should be cleaned or replaced.

Two bags have been supplied. If flow rates are greater than 3000 ml/day, both bags can be connected to the inner ring at the same time by using the tee fitting(Fig. 4) that has been supplied. If the flow rate is less than 3000 ml/day, the extra bag can be filled and weighed in advance so that it can be connected to the inlet tube when the other is removed. By doing this, only one trip to the ring is needed to take a reading.

When connecting a bag to the inner ring, be sure that the valve is closed. If the bag is accidentally lifted out of the water with the valve open, it is possible to lift the inner ring out of the ground or rupture the seal. Each inch of head of water produces an uplift force of about 125 pounds on the inner ring, so holding the bag several inches above the water level with the valve opened can easily lift the inner ring out of the ground.

11.2 WATER LEVEL

The infiltration rate varies with the depth of the water level in the outer ring. For this reason, the water level should be recorded each time a flow measurement is made. Water should be added to the outer ring occasionally in order to keep the water level to within ±1/2 inch of the initial level. A scale taped to the inside wall of the outer ring makes it convenient to monitor the water level.

11.3 TEMPERATURE

The temperature of the water in the rings should be monitored closely for reasons discussed previously. If temperature is monitored with a thermometer, then measurements need to be made as close to the inner ring as possible. The recommended procedure is to put the thermometer in a soda can and then place the can on the ground next to the inner ring. Remove both the can and the thermometer to take a reading. The water in the can should remain at the same temperature as the water near the inner ring long enough to take a reading.

11.4 PURGING AIR FROM INNER RING

During the test, it is possible that air may rise out of the soil and become trapped in the inner ring. This air should be purged from the inner ring and an estimate of its volume made. If the volume is significant (>20% of flow since the last time the ring was purged) the infiltration rate should be corrected to account for it.

The procedure for purging the inner ring of air is described below.

1. Disconnect bag inlet tube. Use a board or shovel handle and gently tap on the inner ring to get the air bubbles to rise to the flush port.

2. Lift the flush tube out of outer ring and lay end of tube on the ground. The end of the tube needs to be below the water level so that water can be siphoned out of inner ring.

3. Remove plug from end of flush tube. Water and air if present will start to flow out of inner ring. If air completely fills the tube, the syphon will be lost. If this happens, submerge end of tube in water of outer ring and work air out of tube. Once the

tube is saturated, place plug in end of tube, lift tube out of ring and place on ground. Remove the plug and allow water to flow from end of tube. (if tensiometers are being used, the hand pump can be used instead to restart the siphon by pulling a vacuum on the end of the tube)

4. Allow water to flow from end of tube until air ceases to emerge from inner ring. Replace plug in end of flush tube and place tube back into outer ring.

5. Wait at least 30 min. before taking any flow measurements.

Purge the inner ring on a weekly basis until no significant amount of air is found.

12.0 DATA REDUCTION

12.1 INFILTRATION

Infiltration (I) can be determined as follows:

$$I = Q/(At)$$

where:

I = infiltration (cm/sec)
Q = volume of flow (cm^3)
A = area of flow (cm^2)
t = time interval in which Q was determined (sec)

Two factors that can have a significant effect on the infiltration rate are temperature changes and swelling of the soil. If either are significant, the infiltration rate should be corrected to account for it. The total flow (Q) that is measured is the sum of the following:

$$Q = Q_i + Q_s + Q_t$$

where

Q_i = flow due to infiltration

Q_s = flow due to swell

Q_t = flow due to temperature changes

The infiltration rate corrected for swell and temperature changes is:

$$I = Q_i / At$$

where:

$$Qi \ = \ Q - Qs - Qt$$

Temperature changes can be minimized as discussed previously and consequently, Qt is seldom significant.

The remaining portion of flow to consider is Qs, the flow due to soil swell. The process of water infiltrating an unsaturated swelling soil is complex and difficult to analyze Presently, there is not an accepted procedure to account for the effect of soil swell on the infiltration rate. The author is of the opinion however, that a close estimate of Qs can be obtained as follows. First, it is assumed that any volume change that occurs is vertical. Second, it is assumed that all of the additional volume generated by the swelling soil is filled with water that infiltrated the soil. Based on these two assumptions:

$$Qs \ = \ \Delta h \ x \ A$$

where

$\Delta h \ =$ vertical swell of soil beneath inner ring

$A \ = $ area of inner ring

12.2 HYDRAULIC CONDUCTIVITY

Hydraulic conductivity (k) in the saturated zone can be determined as follows:

$$k \ = \ Q/(iAt)$$

where:
$Q \ = $ volume of flow (cm^3)
$A \ = $ area of flow (cm^2)
$t \ = $ time interval in which Q was determined (sec)
$i \ = $ gradient
$\quad = \ \Delta h / \ \Delta s$
$\Delta h \ = $ head loss
$\Delta s \ = $ length of flow path for which Dh is measured

since:

$$I \ = \ Q/(At)$$

then:

$$k \ = \ I/i$$

The determination of k depends on calculating a value for the gradient (i). Unlike the calculations for I and k, the determination of i is not straight forward. The parameters used to calculate i for a typical infiltration test are shown in Fig. 5. These parameters are used to calculate the gradient as follows:

$$i \ = \ (H + D + Hs) / D$$

where:

H = depth of water ponded in rings
D = depth to the wetting front
Hs = suction at the wetting front

There are differing opinions on what value should be used for Hs. One view is that Hs should be equal to the ambient suction in the soil below the wetting front. The ambient suction can be measured with tensiometers can be quite high, yielding values of Hs as high as 700 cm of water. Another view is that Hs should be equal to zero, ie. the suction in the soil at the wetting front has no influence on the infiltration rate. Hence, if the position to the wetting front is known, the gradient is simply $(H + D) / D$.

The author feels that the second view (Hs = 0) yields a close approximation to the actual gradient. Measurements made at several sites have shown that the drop in infiltration rate versus time can be accounted for by the increase in D as the wetting front moves through the soil. If suction had an influence, a much larger decrease in the infiltration rate would have occurred.
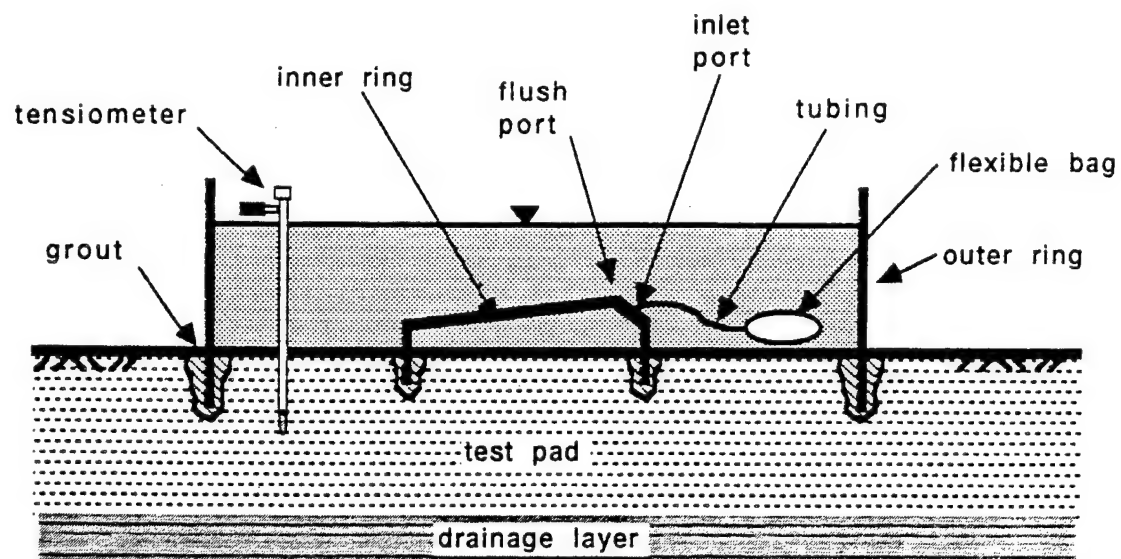
Figure 1. Schematic Of A Sealed-Double Ring Infiltrometer Installed On
Test Pad
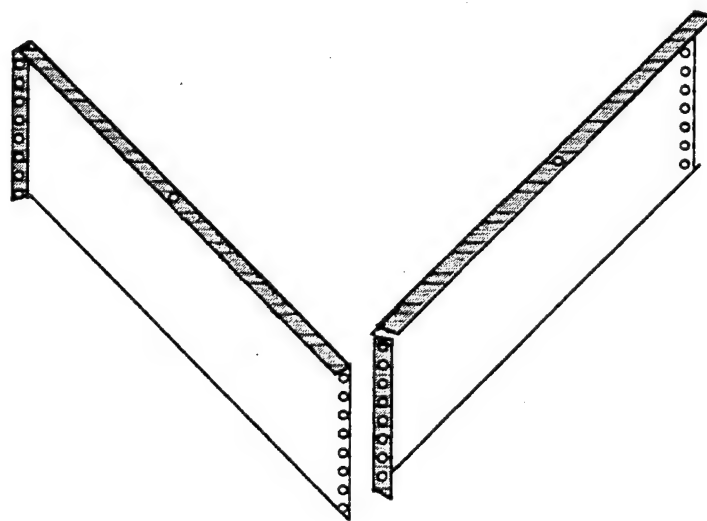
Figure 2. Orientation Of Panels For Assembly Outer Ring.
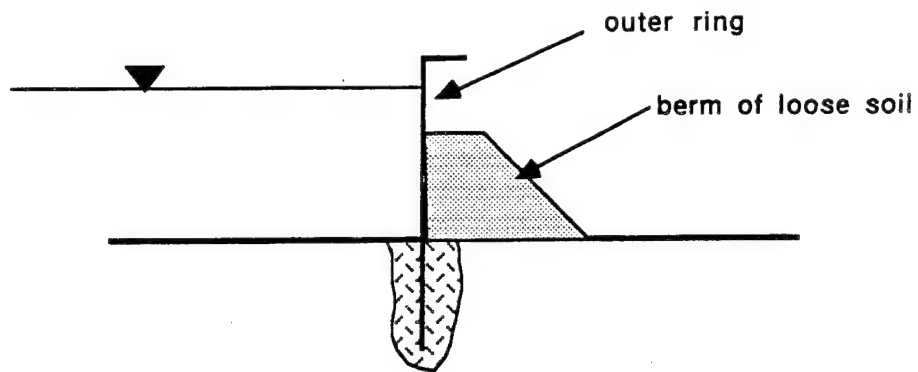
Figure 3.   Berm Of Soil On Outside Of Outer Ring
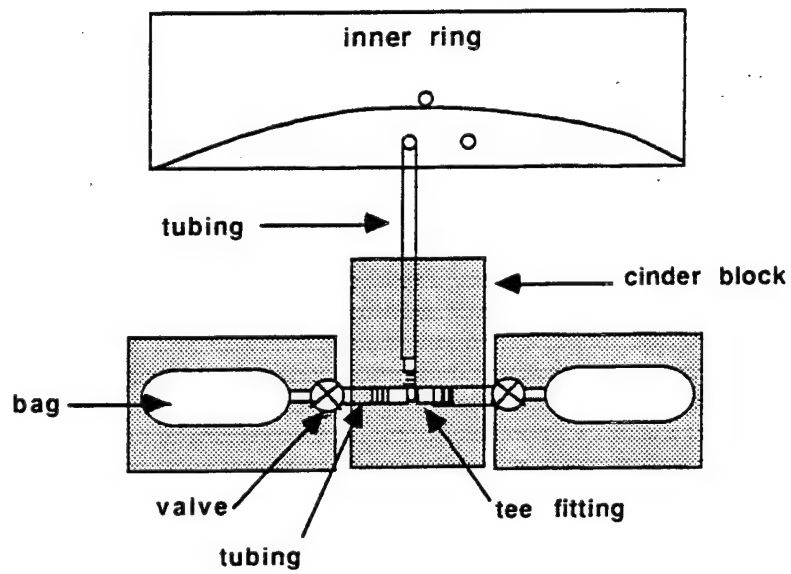
Figure 4.   Arrangement Of Two Bags Connected To Inner Ring
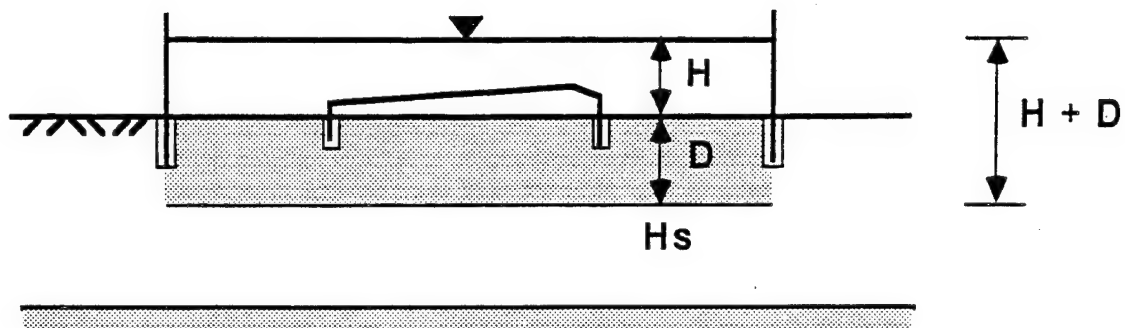
Figure 5.   Diagram With Parameters Needed To Calculate Gradient

APPENDIX A

TENSIOMETER DESCRIPTION AND INSTALLATION INSTRUCTIONS

A schematic of a tensiometer is shown in Fig. 6. It consists of a sealed plastic tube with a porous tip on one end and a vacuum gage on the other. The tube is filled with water and then sealed. Operating instructions have been included with the tensiometers. A preferred method of installation however is described on Fig. 6. Disregard the installation procedure described in the manufacturer's instructions. Driving a pipe may crack the soil and open up flow channels.

Tensiometers work as follows. If the soil is unsaturated and there is good contact between the tip and the surrounding soil, water will be drawn out of the tube and the gage will register a suction. As the wetting front passes the the tip, the suction will decrease and water will reenter the tensiometer until the suction goes to zero. Good performance of tensiometers depends on saturating the tensiometer and achieving good contact between the tensiometer and the surrounding soil.

It is recommended that nine tensiometers be used, three at each depth of 6", 12", and 18". A suggested layout for the tensiometers is shown in Fig. 7. Auguring a hole and pushing the tensiometer in place is preferred to forming a hole by driving a pipe into the ground.

All dimensions shown are for
tensiometer installed at the 6
inch depth

° Add 6 and 12 inches to
these dimensions for the
tensiometers installed at
12 and 18 inches respectively

1. Auger a 2-3/4" diameter hole
   with a hand auger.

2. Use a 7/8" auger bit advance
   hole 3-1/4".

3. Push tensiometer into hole

4. Compact a mixture cuttings
   removed in augering process
   mixed with bentonite into the
   annulus space.

24"

12 1/8

cuttings mixed with bentonite
compacted into annulus

4""

6"

7 1/8"

1 1/8

2 1/4

7/8

Figure 6.   Schematic of a Typical Tensiometer Installation

APPENDIX B

SWELL MEASUREMENTS

An arrangement that can be used to measure swell is shown in Fig. 8. This arrangement consists of stretching thin wire between two fence stakes that have been driven into the ground close to the outer ring. The stakes are aligned so that the wire is positioned directly above two handles on the inner ring. The wire serves a a reference elevation to monitor upward movement of the inner ring. A swell gage is used to establish a reference distance between a reference mark on a handle and the wire. The swell gage consists of a dial gage mounted to a rod. The position of the dial gage on the rod is fixed.

It is recommended that two wires be used so that the elevation of all four handles on the inner ring can be monitored. A olt can be attached to the handle by driliing a 3/8" hole through the handle. The bolt head can serve a reference point for the end of the swell gage rod. Initial readings are taken before the rings are filled to serve as a reference. It is also recommended that the elevation of the wires be surveyed and referenced to a benchmark located away from the rings. If a discrepancy occurs in the readings, the wire can be surveyed to check that its position has not changed.
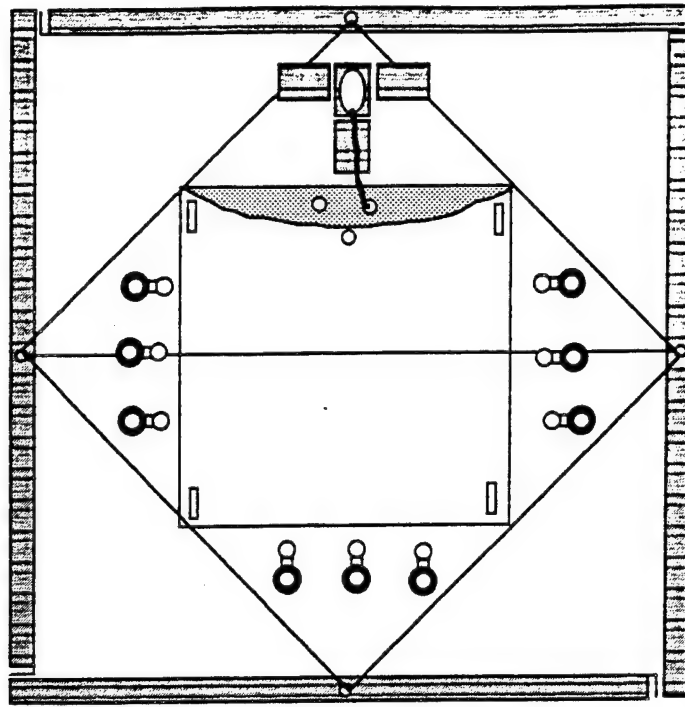
Figure 7. Top View Of SDRI Showing Layout Of Tensiometers
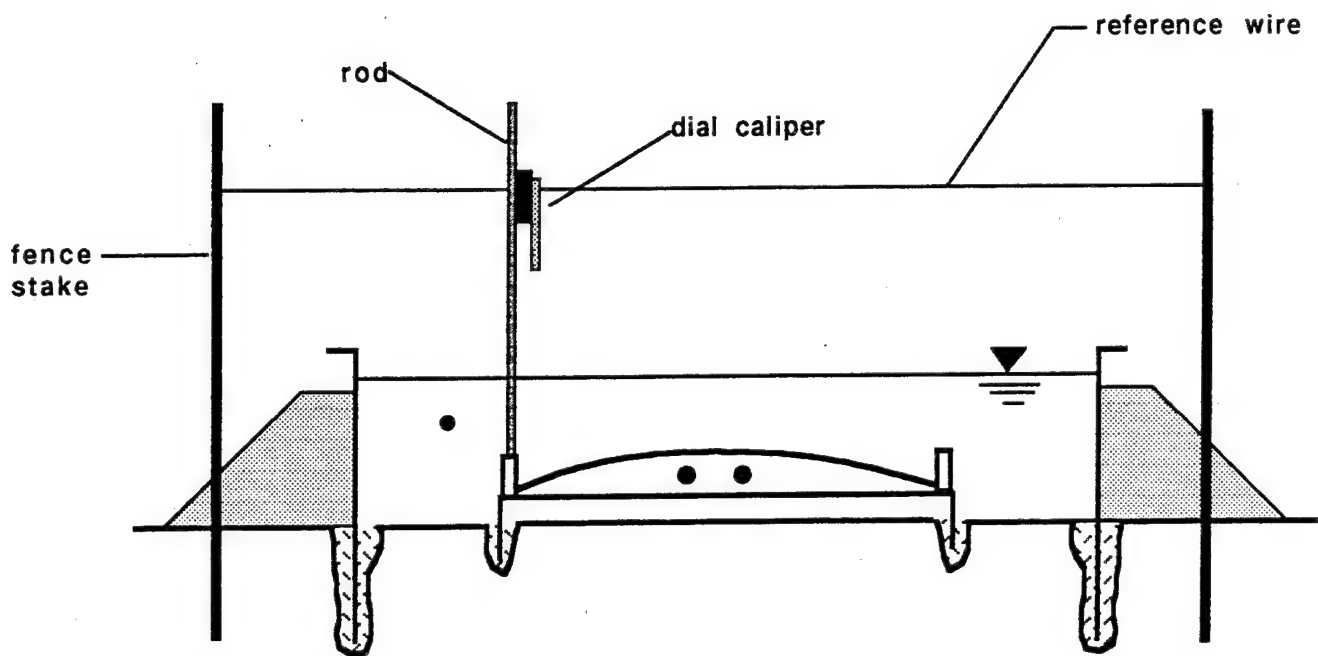
Fig. 8. Arrangement For Monitoring Swell

APPENDIX C

COVER DESCRIPTION

It is highly recommended that a cover which insulates the rings and blocks sunlight is be used during the test. A cover consisting of plywood, 2 x 4's, and insulation is shown in Fig. 9.

COVER MATERIAL LIST
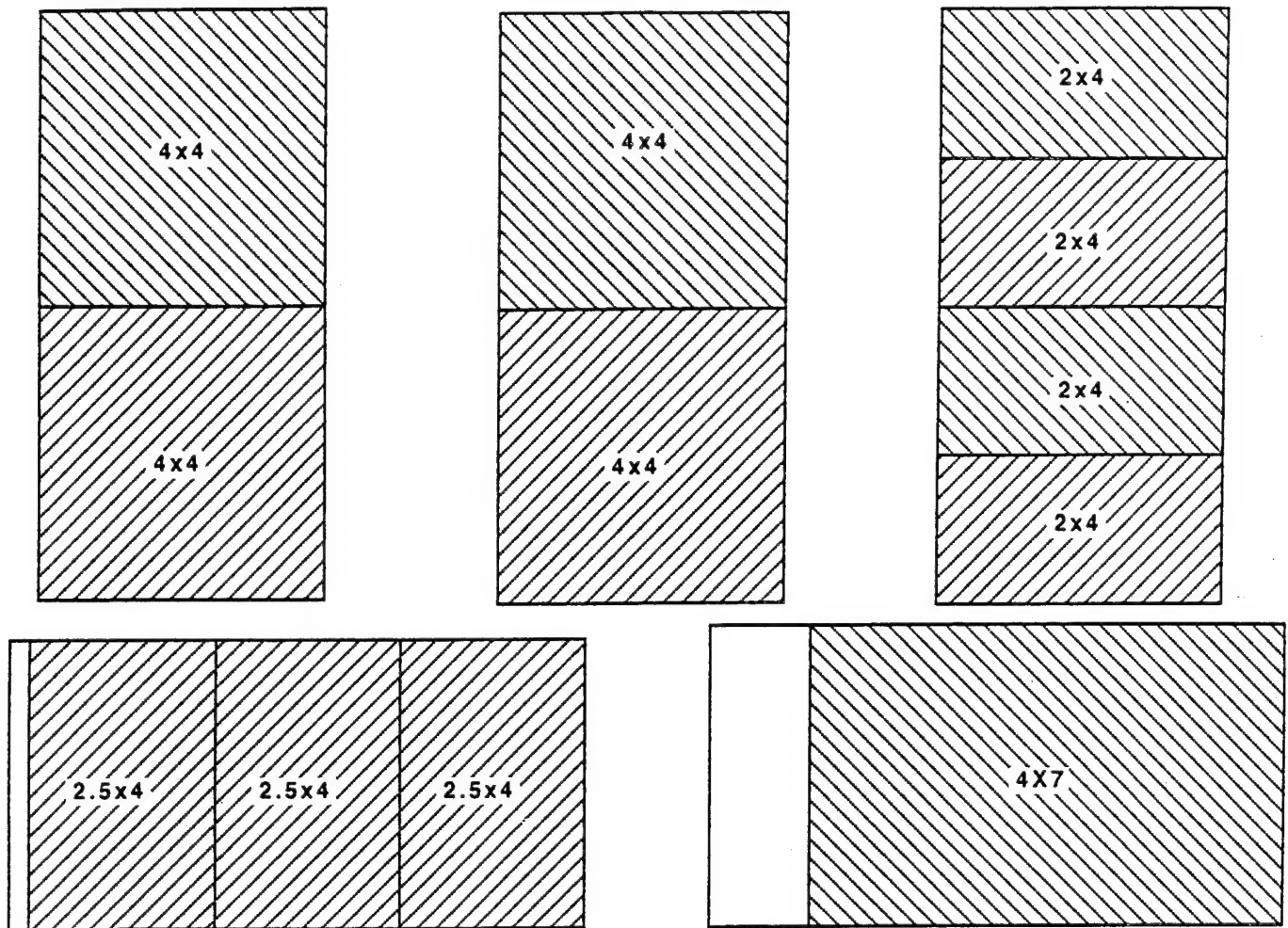
MATERIALS

nails	2x4's	1/2" plywood	insulation
# 16 common 2" roofing	13 - 12'	(5) sheets cut as follows: 4 - 4x4 4 - 2x4 1 - 7x4 3 - 2.5x4	5 - 4x8x1/2 sheets (foil backed insulation- the type used on ext. walls of houses)

Fig. 9a. SDRI Cover

TOP VIEW OF SDR1 COVER
WITH 2X4'S IN PLACE

TENSIOMETERS

INNER RING

FLEXIBLE BAG

Fig. 9b. SDR1 Cover

TOP VIEW OF SDRI COVER
WITH PLYWOOD IN PLACE
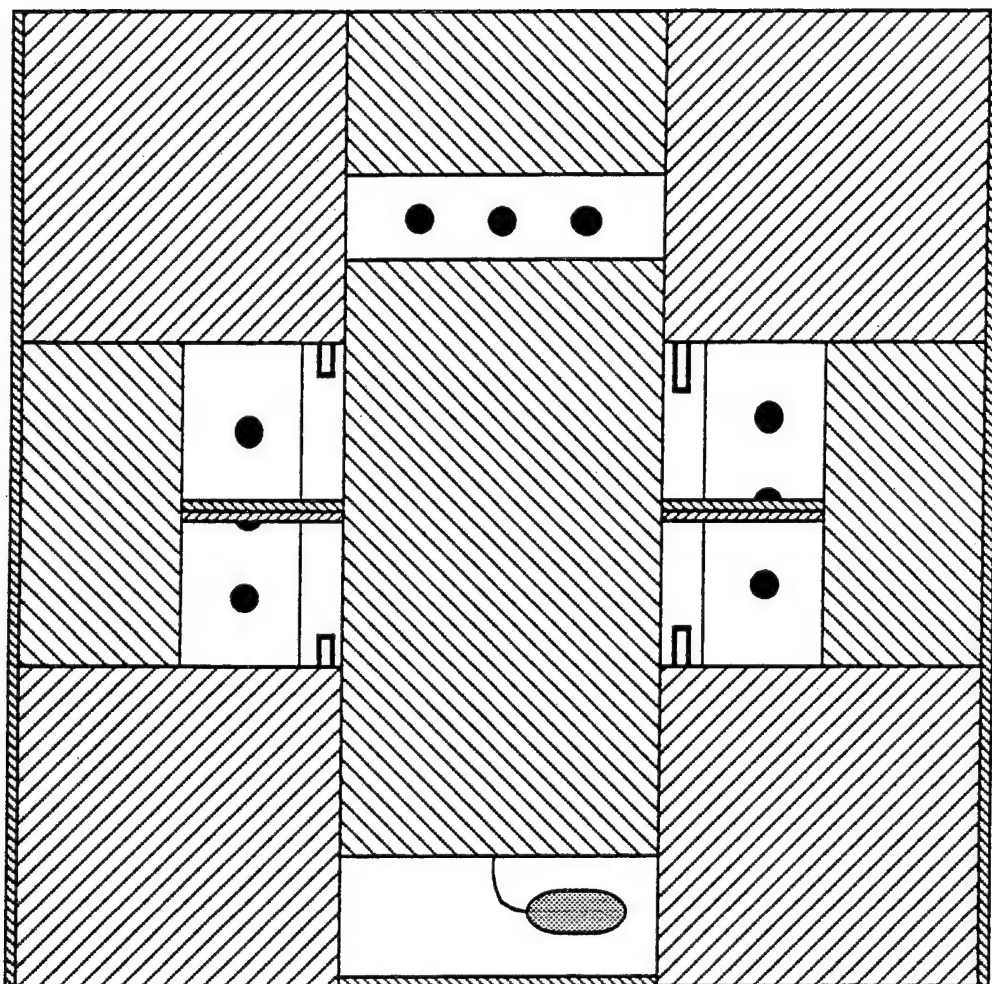
Fig. 9c. SDRI Cover

APPENDIX D

DATA FORMS

SWELL DATA

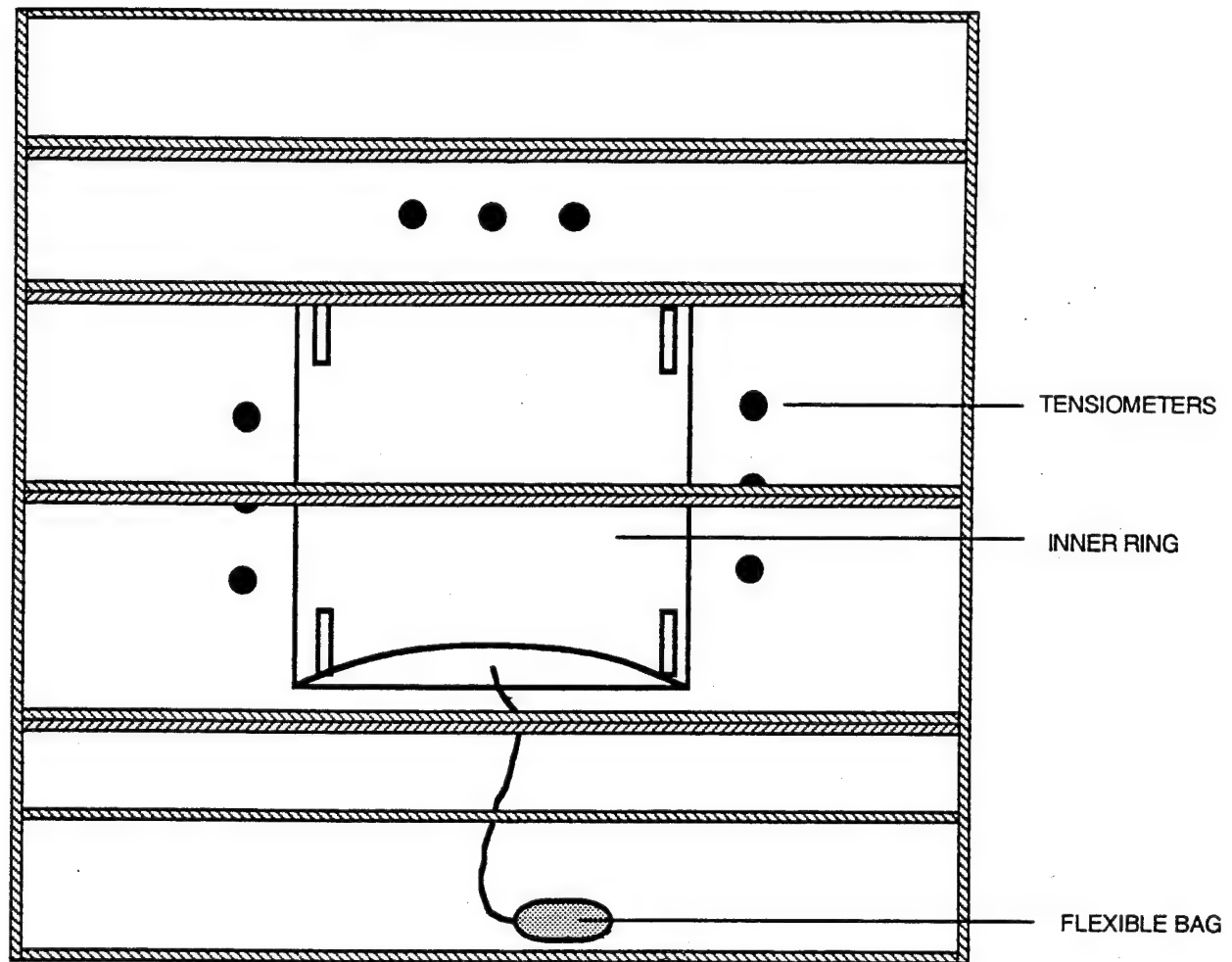| DATE	TIME	1	$\triangle h$	2	$\triangle h$	3	$\triangle h$	4	$\triangle h$	$\triangle h_{avg}$

NOTES

TENSIOMETER READING – CENTIBARS

DATE	TIME	DEPTH – 6"				DEPTH – 12"				DEPTH – 18"			
		GROUP 1	GROUP 2	GROUP 3	AVG.	GROUP 1	GROUP 2	GROUP 3	AVG.	GROUP 1	GROUP 2	GROUP 3	AVG.

NOTES:

PROJECT: _____

TENSIOMETER TYPE: _____

DATA FORM FOR SDRI TEST

Project: _____
Inner ring Info: _____
Outer ring Info: _____
Liner thickness (cm) : _____

$$I = Q/(At)$$

I = infiltration (cm/sec)
Q = quantity of flow (ml)
t = interval of time (sec)
A = area of inner ring (cm 2)

Date	Initial time (sec)	Final Time (sec)	Interval of • Time (sec)	Initial Wt. of Bag (grams)	Final Wt. of Bag (grams)	Q Quan. of Flow (ml)	I Infiltra-tion (cm/sec)	Temp. (C)	Water Depth (in)
on off								on off	on off

NOTES:

Appendix B

THE STEI TWO-STAGE BOREHOLE FIELD PERMEABILITY TEST

THE STEI TWO-STAGE BOREHOLE
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SYNOPSIS

In 1991, the Texas Department of Health began requiring field verification of the hydraulic conductivity for the waste-retention barriers under its jurisdiction. The TDH has approved two procedures: the Two-Stage Borehole (TSB) method and the Sealed, Double-Ring Infiltrometer (SDRI) method. The TSB method is discussed herein.

It is a falling-head infiltration test conducted in a cased borehole, typically 4 inches in diameter. The first stage is performed with the bottom of the hole flush with the bottom of the casing for maximum effect of vertical permeability (k_v). After steady-state is achieved, the hole is advanced some 6 to 8 inches below the bottom of the casing so that horizontal permeability (k_h) has a greater effect. The two stages yield the following:

Stage 1 - The maximum possible value for (k_v).
Stage 2 - The minimum possible value for (k_h).
Stage 1 + Stage 2 - Constants for two equations which can then be solved for the real (k_h, k_v).

Procedures are available for reduction of the data in the cases of both above and below water table testing, and for the bottom boundary conditions of a material far more permeable, equally permeable, or far less permeable than the medium being tested. The test has been successful in evaluating both compacted and natural materials with permeabilities as low as 1×10^{-9} cm/sec.

The major test precautions include proper sealing of the casing along the outside, accounting for temperature effects, and correcting for sidewall smear during the second stage. The test is quick, simple, and relatively inexpensive. It allows results in days, rather than months. Multiple installations are feasible so that statistical confidence can be achieved. It is recognized in the literature, including U.S. EPA publications, and accepted by many State regulatory authorities.

I. INTRODUCTION

Clay barriers are an important component of waste retention structures. Their primary geotechnical characteristic for this use is hydraulic conductivity, which must be verified during the Construction Quality Assurance program. Until recently, practice relied on laboratory testing of small (7 to 10 cm diameter) undisturbed samples taken from the barrier or a similarly constructed test pad. Day and Daniel (1985) reported conductivities measured in the field which were 3 to 4 orders of magnitude higher than they obtained with laboratory tests. While that study was justly criticized, the horse was out of the barn and regulators all over the country galloped into field testing for hydraulic conductivity evaluation at waste facilities.

From the regulatory standpoint, a test procedure should be accurate and avoid false positives, i.e., not indicate compliance with the specified conductivity when the liner or pad truly has a higher value. This normally means testing a large soil volume searching for the elusive "macropores" which are thought to evade, somehow, even numerous laboratory tests. The regulated community wants the accuracy and avoidance of false positives for their own protection, but also wants to minimize testing times (and costs), and to avoid false negatives, both for economy.
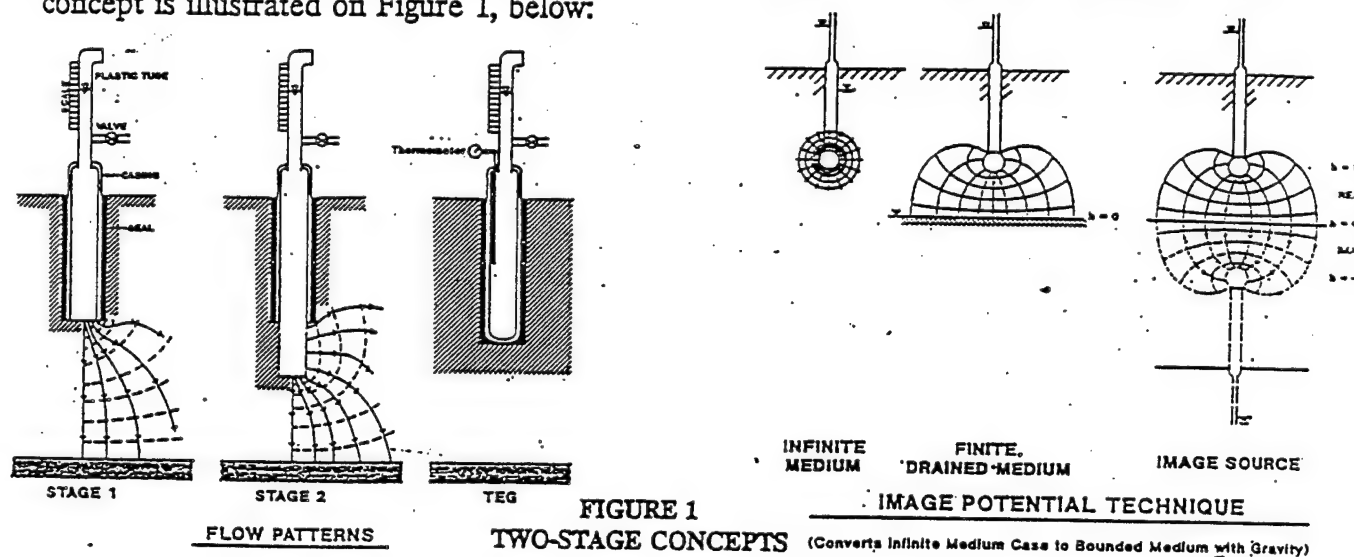
In about the last two years, two methods have become accepted as meeting these criteria to a satisfactory degree: the Sealed, Double-Ring Infiltrometer (SDRI) and the Two-Stage Borehole (TSB) procedure. Each has its stronger and weaker points (see Daniel, 1989).

II. BASIC CONCEPTS

The vertical conductivity (k_v) governs flow, even in sidewall liners if built in the preferred manner: lifts parallel to the slope. However, the horizontal conductivity (k_h) is greater than the vertical. All field tests are affected by this anisotropy, unless flow in the horizontal direction is artificially blocked; the effect is to increase the test conductivity by factors of 2 to 5 over the real (k_v) value. Equations for flow from various source geometries in a cross-anisotropic medium are available in Hvorslev (1951). However, each equation has two unknowns: (k_v) and ($m^2 = k_h/k_v$).

The TSB procedure combines four old concepts into one new idea to find (k_v). The field procedure is taken from long-established US Bureau of Reclamation methods: their flush-bottom borehole test (E-18) and borehole packer test (E-19). Computations are based on the Hvorslev equations adapted for various bottom boundary conditions by the three-dimensional Image Potential Technique (Carslaw and Jaeger, 1959). The new idea is performing both USBR tests in the same borehole, yielding two equations which can be solved for the two unknowns, (k_h) and (k_v).

The TSB is a field infiltration test, conducted in a cased borehole so that the geometry of the infiltrating zone can be controlled. It is normally conducted as a falling-head test. The basic idea is to vary the geometry of the infiltrating area so as to vary the relative effects of (k_h) and (k_v). In the first stage, the geometry is chosen so that (k_v) has its maximum effect. The second stage geometry is such that (k_h) has its maximum effect. The results of the two stages yield two equations in two unknowns (k_h, k_v), which can then be solved. This concept is illustrated on Figure 1, below:

FIGURE 1

FLOW PATTERNS   **TWO-STAGE CONCEPTS** (Converts Infinite Medium Case to Bounded Medium with Gravity)

Stage 1 is normally conducted using a flat bottom flush with the base of the casing. Infiltration proceeds until a steady-state flow condition is achieved. Then, the borehole is advanced some 1.5 to 2 casing diameters (6 to 8 inches) below the bottom of the casing. The apparatus is refilled, and infiltration in this Stage 2 continues until it achieves steady-state flow.

During the test, the soil is assumed isotropic $(k_v = k_h)$. Stage 1 then yields an apparent permeability (K1), and Stage 2 a different value (K2). The unknown ratio (k_h/k_v) is a unique function of the known test geometry and the known test ratio (K2/K1). When the former is determined, the real (k_v, k_h) can be computed from (K1) or (K2).

III.   FIELD PROCEDURES

As is the case with virtually all field tests, and especially field permeability tests, the field procedures are of paramount importance. The most diligent office analyses cannot overcome all of the problems resulting from improper installation, inadequate monitoring, premature test termination, and the like.

3.1 Test Program Design. The test program should be designed to meet the conditions assumed in deriving the data reduction equations so that meaningful results can be obtained.

3.1.1 Vertical Boundaries. Certain clearances are required between the infiltrating surface and any boundaries, pervious or impervious. These can be summarized as:

a. Minimum casing embedment below ground surface = 2.5D
    (Prevents uplift, minimizes hydraulic fracturing)

b. Minimum thickness of tested material below bottom of Stage 2 = 2.0D
    (Avoids violating boundary conditions of equations)

c. Minimum recommended Stage 2 extension = 1.5D
    (Avoids theoretical problems at finite but small L/D)

    D = Casing inside diameter
    L = Length of Stage 2 extension

3.1.2 Horizontal Spacings. It is intuitively obvious that the tests must be spaced "far enough" apart so that their flows do not interfere with each other causing a falsely low permeability. Also, the presence of a drainage boundary (such as the edge of a test pad) which is "too close" to the test will increase the flow, yielding a falsely high permeability.

This can be avoided by maintaining at least the following clearances:

a. Minimum horizontal distance between tests = 30D
b. Minimum horizontal distance to free surface = 30D

3.1.3 Number and Size of Tests. The number of tests required for evaluation depends on the project, the acceptance criteria, and the variability of the stratum/fill being evaluated. As in virtually any other geotechnical testing, "the bigger the better".

However, the general practice has been to use 4-inch (ID) tests, with 5 tests for the typical liner or test pad.

The scale effect, if any, of test size has not been fully researched. Virtually all of the known tests have been conducted using 4-inch (10 cm) ID casings. These tests typically permeate a volume of some 0.4 - 1.1 cubic feet each, or 2 to 5 cubic feet for a 5-test group. Benson (pers. comm., 1991) indicates that the minimum representative volume for a permeability determination is on the order of 0.5 - 1.0 cubic foot. This is about the volume permeated by a typical TSB test.

3.1.4 Other Details. There are a few other details in test planning which should be considered. Among these are:

a.   Protect the test area surface from desiccation, usually with <u>clear</u> or <u>white</u> plastic.
(Avoids heat-induced problems).

b.   Use a "sock" to prevent collapse of the Stage 2 open hole in susceptible materials. The sock is a rigid cylinder of open-mesh plastic, lined with a filter geofabric. The cylinder is somewhat smaller in diameter than the casing ID (and thus the Stage 2 hole), and an inch or so longer than the extension for Stage 2. It is fitted with retrieving lines and not left in the hole after the test.

c.   Minimize the distance (R_z) from the ground surface to the bottom of the measuring scale, especially for shallow tests. This also aids in having the longest possible reading time between standpipe refills and avoiding hydraulic fracturing.

d.   Match the standpipe size to the flow rate so that accuracy is achieved but overnight readings are possible. For a 4-inch casing, this usually means a 0.5 - 0.75 inch ID standpipe.

<u>3.2 Permeameter Installation.</u> Proper installation and checking the permeameters are vital to obtaining a valid test. Various field techniques have been developed through experience which minimize problems. These techniques are discussed in this section.

<u>3.2.1 Permeameter.</u> A typical permeameter is illustrated on Figure 2. The apparatus is simple; the permeameter can be assembled with a visit to a water-well driller and a hardware store. The elements for a falling-head system are:

a.   <u>Casing.</u> Typically 4-inch ID Schedule 40 PVC monitoring well pipe, flush-threaded, with "O"-Ring joint. Other casings can be used.

b.   <u>Cap.</u> To fit casing, preferably domed, and drilled and/or tapped to receive the standpipe apparatus.

c.   <u>Standpipe.</u> *Clear* Schedule 40 PVC or acrylic tube, 0.5 to 1.0 inch ID, with scale. Include elbow with cover (having air-vent) to prevent rain entry and minimize evaporation.

d.   <u>Fittings.</u> The small fittings necessary to assemble the apparatus.

All joints which are not glued are assembled with PTFE Plumber's Tape and silicone grease (<u>not</u> sealant).
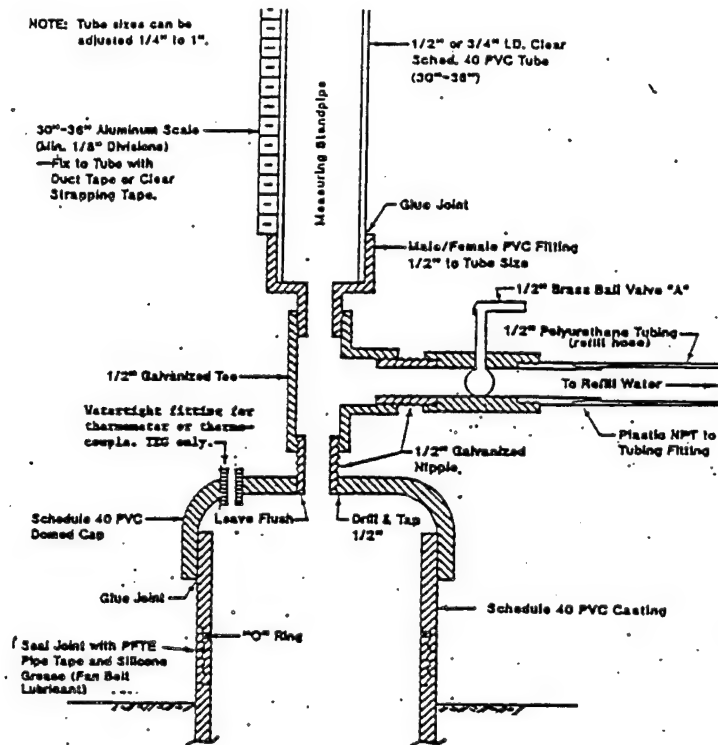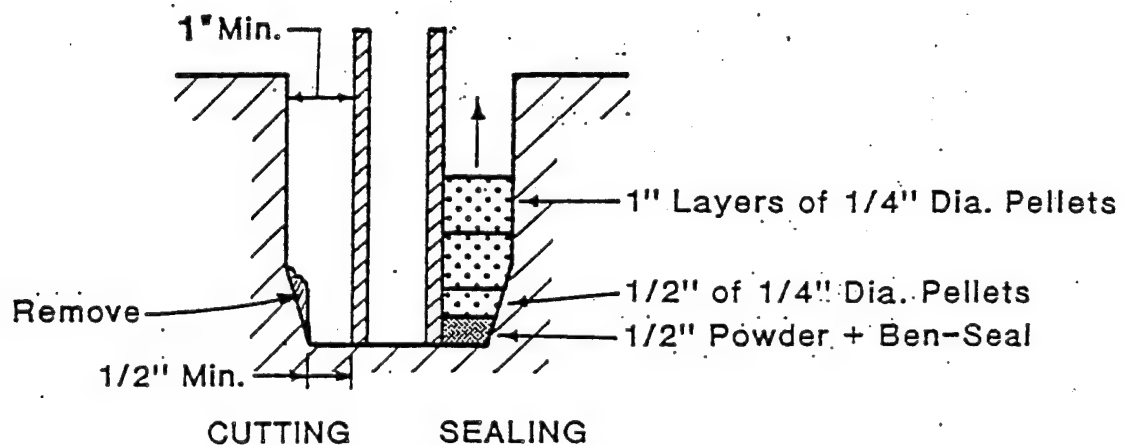
NOTE: Tube sizes can be
adjusted 1/4" to 1".

Measuring Standpipe

1/2" or 3/4" I.D. Clear
Sched. 40 PVC Tube
(30"-38")

30"-36" Aluminum Scale
(Min. 1/8" Divisions)
—Fix to Tube with
Duct Tape or Clear
Strapping Tape.

Glue Joint

Male/Female PVC Fitting
1/2" to Tube Size

1/2" Brass Ball Valve "A"

1/2" Polyurethane Tubing
(refill hose)

1/2" Galvanized Tee

To Refill Water

Watertight fitting for
thermometer or thermo-
couple. IEG only.

1/2" Galvanized
Nipple.

Plastic NPT to
Tubing Fitting

Schedule 40 PVC
Domed Cap

Leave Flush

Drill & Tap
1/2"

Glue Joint

Schedule 40 PVC Casing

Seal Joint with PFTE
Pipe Tape and Silicone
Grease (Fan Belt
Lubricant)

"O" Ring

FIGURE 2
TYPICAL PERMEAMETER

1" Min.

1" Layers of 1/4" Dia. Pellets

Remove

1/2" of 1/4" Dia. Pellets

1/2" Powder + Ben-Seal

1/2" Min.

CUTTING   SEALING

FIGURE 3
SEALING PROCEDURE

3.2.2 Borehole. The casing is set into a borehole. The holes have been drilled using rigs, power-operated hand equipment, and hand augers. The device depends on depth and hardness of material. The hole must have a large enough diameter to allow sealing the annular space between the borehole wall and the casing. Also, it must not disturb the soils below the casing bottom. The bottom must also be flat. Experience has shown the following to be acceptable:

a.   Borehole diameter at least 2 inches greater than the casing OD. (To allow sealant to reach the bottom and for tamping).

b.   Stop point of auger about 1 inch above proposed casing bottom. (To avoid testing in a disturbed material)

c.   Ream bottom of borehole to final depth with a flat auger.

The bottom of the borehole should be flat and flush with the bottom of the casing in order to correspond with Hvorslev's (or H-I) Case "B" or "C" for Stage 1.

3.2.3 Sealing. This is the single most important step in installation. A poorly sealed test cannot be salvaged. The annular space between the casing and the wall of the borehole is sealed with bentonite. Best results have been attained using 1/4" (not 3/8" or larger) bentonite pellets or crushed bentonite (Baroid "Hole-Plug" or equivalent). The procedure, illustrated on Figure 3, is:

a.   Crush sufficient pellets, "Ben-Seal", or "Hole-Plug" to fill about 1/2 inch of the annulus. This should have about 1/16" size fragments with some powder.

b.   Place this material into the annular space.

c.   Place about 1/2 inch of bentonite pellets or "Hole-Plug" into hole,

d.   Tamp the bentonite pellets or "Hole Plug",

e.   Add water until it shows above the bentonite,

f.   Repeat the process (but using only the pellets or "Hole Plug") in 1 inch increments to the ground surface or a minimum of 6D above the casing bottom, whichever occurs first. Grouting above the 6D level is allowable.

g.   Allow the bentonite (and grout) to hydrate at least overnight.

The casing must be steadied to prevent lateral motion while sealing. The bentonite seal is then allowed to hydrate overnight before any head is applied to the system.

3.2.4 Advancing for Stage 2. Upon completion of Stage 1, a borehole is advanced below the bottom of the casing to form the cylindrical infiltrating surface for Stage 2. The important points are:

a.   Do not disturb the casing - that can affect the seal.

b.   Borehole diameter should equal casing ID.

c.   Stop point of auger about 1 inch above proposed Stage 2 bottom. Ream flat and measure depth.

d.   Roughen the sidewalls to minimize smear.

This portion of the work is normally handled with hand equipment. The first step after removing the cap is to empty the casing of water (tests above groundwater level or where no seepage was noted during Stage 1 drilling and/or sealing). It is frequently useful to obtain an undisturbed sample during this process, using ASTM D2937 or D1587. However, undisturbed sampling should not be performed if the material being tested contains gravel-sized particles; they can disturb the sidewalls during the push or driving. After or in lieu of undisturbed sampling, the boring is augered until the point of the auger is about 1 inch above the desired bottom for Stage 2. The auger should be at least 1/2 inch in diameter smaller than the casing ID. The boring is then completed to depth and diameter with a flat-bottomed reamer.

The reamer is designed to minimize sidewall smear, having full casing ID only at the cutting edge. The sidewalls are then roughened with a wire brush or similar device, a procedure also recommended in USBR E-18. This step must not be omitted, since one of the significant problems encountered in Two-Stage testing has been artificially low values for Stage 2 due to smear. Equations to handle smear are included herein, but require some idea of the degree of smear.

After the borehole is completed and cleaned of cuttings, the depth is measured so that the correct length of the Stage 2 cylinder is known. For a typical test, a 1-inch depth error will yield the wrong Stage 2 permeability value by 7 to 8%. The cap is then reseated, and Stage 2 begins.

3.3 Ambient Condition Effects. Temperature changes cause the dominant effects of ambient conditions on this test, although there may be some contribution from barometric pressure changes. Temperature changes affect the test by:

* Volumetric changes in the water and apparatus.
* Viscosity changes with temperature.
* Freezing the test water.

The procedures for overcoming these effects are given below.

3.3.1 Volumetric Effects (TEG). At slow rates of flow, the field readings are affected by temperature, as has been noted on many such projects. Rising temperature causes the water column in the pressure/measurement standpipe to expand, so that the drop in water level is less than flow alone would produce. The net effect is a lower apparent permeability. Conversely, falling temperature produces a higher apparent permeability. A normal day's temperature variations can easily cause a 0.5 to 1 order of magnitude change in the apparent permeability of low-permeability materials.

Therefore, a complete "dummy" test setup is installed but with the bottom of the casing sealed with a cap which is normally glued on and pressure-tested. This dummy, or temperature effect gauge (TEG) is of the same construction and embedded to the same depth as the regular test setups. Since there is no flow from the TEG, any change in its readings must be due to changes in the ambient conditions (temperature and/or barometric pressure). Such changes would affect the regular test setups to exactly the same degree.

This correction is applied to the regular tests by:

* Reading the TEG at the same times as readings are taken on the regular tests.

* Determining any increase (decrease) in water levels in the TEG between regular test readings.

* Subtracting any increase (adding any decrease) at the TEG from the readings at the regular tests for the ends of the same time increments.

3.3.2 Viscosity Effects of Temperature. Permeability is normally reported as the value for water at 20°C (68°F). The density and viscosity of liquids, including water, are affected by temperature. The effect on permeability is in direct ratio to the kinematic viscosity (U), which is the viscosity divided by the density. The kinematic viscosity decreases at higher temperatures. The net effect is that the apparent permeability is greater than the 68°F value at low temperatures. The reverse occurs with decreasing temperatures. The effects for ordinary conditions can be from -50% to +15% on the permeability value. The normal correction to the standard condition is given in ASTM D5084:
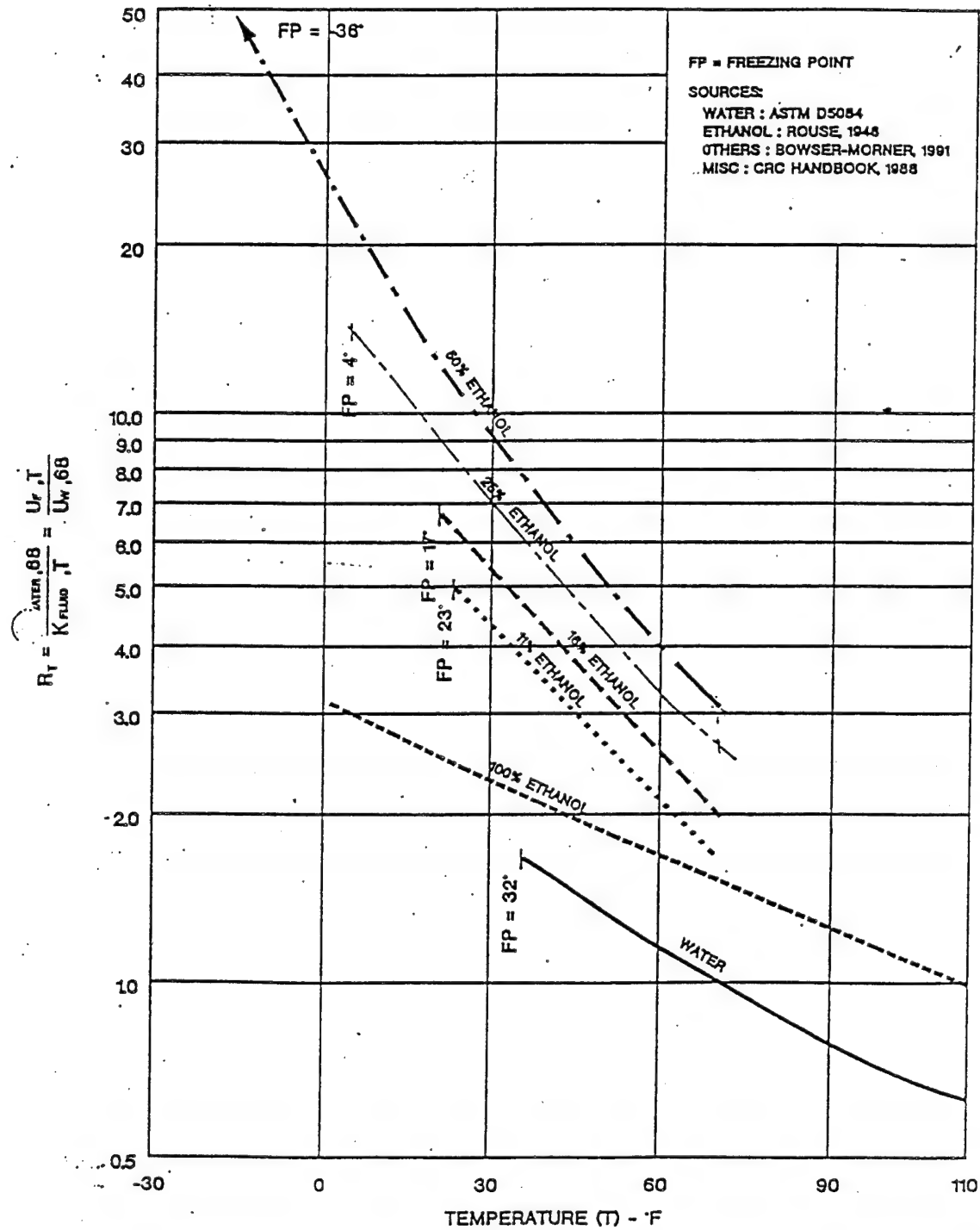
FIGURE 4
KINEMATIC VISCOSITY FACTORS

$$k_s/k_T = R_T = U_T/U_s \qquad (3.3.-1)$$

where: k_s =   Permeability for water at 68°F

k_T =   Permeability observed in test

R_T =   Correction Factor

U_T =   Kinematic viscosity for test fluid at test temperature

U_s =   Kinematic viscosity for water at 68°F

The factors (R_T) are given for water at temperatures between freezing and 120°F (0 to 49°C) in ASTM D5084. See also Figure 4 which reproduces that data.

The temperature of the exfiltrating water is measured by a thermometer or thermocouple in the TEG. It should extend to roughly the bottom of the casing. The thermometer or the leads for the thermocouple should have its own (sealed) port into the TEG cap or casing. Running either through the TEG standpipe could easily affect its function of volumetric correction.

3.3.3 Freezing Conditions. Unfortunately, field testing must sometimes proceed when the air temperatures are below freezing. Landfill operators often complete a test pad in late fall, so that they will have approval from the regulators for construction in the spring. Even if the ground temperatures stay above freezing, one cannot get decent readings from a frozen standpipe. Three procedures have been used:

a.   **Insulate the exposed test equipment, exposing only to make readings.**
(Only if mean daily air temperature exceeds freezing)

b.   **Use an antifreeze.**
(Ethanol as Vodka is good, but needs its own R_T vs temperature graph. Does not attack clay at 25% or less alcohol).

c.   **Heat the test units.**
(Potential for different temperatures - can invalidate the TEG).

3.4 Conducting the Test. The following discussion is applicable to both Stage 1 and Stage 2. Basically, the procedure is:

a.   **Fill and assemble permeameters.**
(Use PTFE tape and silicone grease. Pour slowly to avoid bottom erosion).

b. Read standpipe levels over time at the permeameter, plus level and temperature at the TEG.
(Levels: to 1/16", temperature to 1°F).

c. Convert these readings to apparent permeabilities.

d. Continue the test until these permeabilities remain steady.

3.5 Field Calculations - Apparent Permeability. The data from each reading is converted into an apparent permeability, termed (K1) for Stage 1 and (K2) for Stage 2. Keeping up with the data in terms of a permeability has a physical meaning, and also yields a better "feel" for the behavior of the medium being tested. If that medium were isotropic ($k_h = k_v$), then (K1,K2) would be "the" permeability. Remember that the objective of most field permeability tests on regulated facilities is to determine that the vertical permeability (k_v) of the liner is not greater than some value, usually 1x10(-7) cm/sec, or to show that the horizontal permeability (k_h) of a drainage material is not less than some value, typically 1x10(-2) cm/sec. It can be shown that (K1) is the maximum possible value for (k_v) and that (K2) is the minimum possible value for (k_h). Hence, using these apparent permeabilities (K1,K2) frequently allows "pass-or-fail" determination early in the testing process. For example, (K1<Spec) within 24 hours in 90% of tests where (k_v/Spec<0.6), and 70% of all tests.

The equations for both Stage 1 and Stage 2 follow the generic falling-head test format:

$$k = R_T G \, Ln(H_1/H_2)/(t_2 - t_1) \qquad (3.5\text{-}1)$$

where: k = Permeability
H_1 = Initial head (at $t=t_1$)
H_2 = Final head (at $t=t_2$)
t_1 = Initial time
t_2 = Final time
G = Geometric Constant, depends on test geometry
R_T = Kinematic viscosity correction to water at 68°F

In both Stages, the head is taken as the distance from the level in the standpipe to the groundwater level. The distance from the bottom of the casing to the groundwater level is limited for calculation purposes (only) to no more than 20 times the casing ID. If the depth to groundwater is less than 20 times the casing ID, the true depth is used in the calculations. However, where the depth to groundwater exceeds this criterion, it is considered to be at this 20-diameter depth in the calculations. This limitation is derived by 3-dimensional analogy with the two-dimensional "effective radius" of a well. The volumetric effects of temperature are accounted for using a corrected final head, replacing (H_2) by (H_2'), where:

$$H_2' = H_2 - c \qquad (3.5\text{-}2)$$

where: c   =   Increase in TEG standpipe water level during time period from t_1 to t_2

If the TEG standpipe water level goes up between readings, (c) is positive and ($H_2' < H_2$). Conversely, (c) is negative and ($H_2' > H_2$) if the TEG standpipe level drops between readings. This step is not theoretically precise, but is close enough for test purposes. The theoretical solution yields a complex implicit equation in which the true permeability is a function of its own logarithm. However, for the geometry of the test setups and the observed magnitudes of increases/decreases, the apparent permeabilities calculated in this manner differ from the true permeabilities by no more than 2 to 5 percent. The net result is to "smooth" the apparent permeabilities. This smoothing is most apparent (and most useful) when the soil's apparent permeability is less than about 2 to $5\times10(\text{-}7)$ cm/sec and especially for small-diameter standpipes.

The kinematic viscosity factor (R_T) used in the calculation is that for the <u>average</u> test water temperature during the period from (t_1) to (t_2).

    3.5.1 Stage 1. The nomenclature for the various terms of the Stage 1 calculations is illustrated on Figure 5. The proper equation is given below; it is the solution for (k_v) for an isotropic medium ($k_h/k_v = 1$).

$$K1 = R_T(\pi d^2/11D_1)[1 + a(D_1/4b_1)]Ln(H_1/H_2')/(t_2\text{-}t_1) \qquad (3.5\text{-}3)$$

where:
     d   =   ID of Standpipe
     D_1 =   Effective diameter of Stage 1 (Casing ID or OD)
     b_1 =   Depth of tested medium below bottom of casing
     a   =   +1 for impervious lower boundary
     a   =   0 for infinite depth of tested medium ($b_1 = \infty$)
     a   =   -1 for pervious lower boundary

And the other terms are as defined above. For field use, the geometric terms are combined into a single constant:

$$K1 = R_T \, G1 \, Ln(H_1/H_2')/(t_2\text{-}t_1) \qquad (3.5\text{-}4)$$

where: G1   = $(\pi d^2/11D_1)[1 + a(D_1/4b_1)]$

A complete example is given in the Sample Calculations, Appendix A.
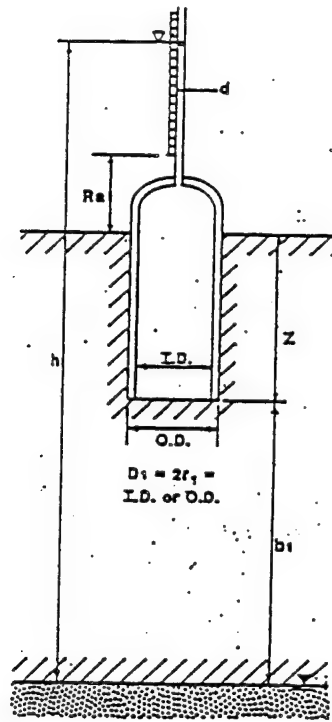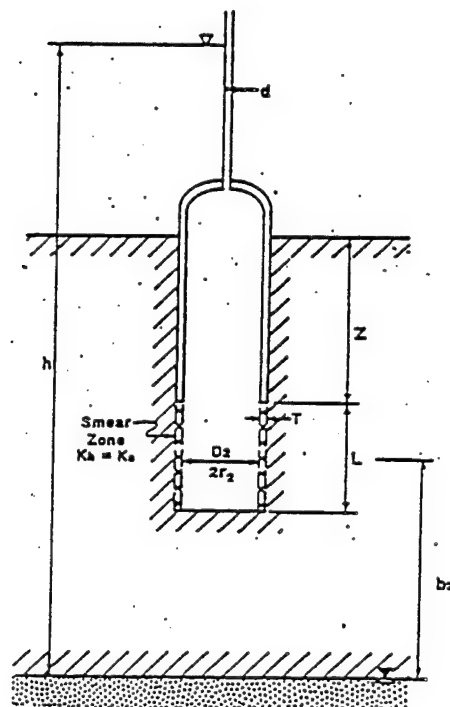
FIGURE 5
STAGE 1 NOMENCLATURE

FIGURE 6
STAGE 2 NOMENCLATURE

3.5.2 Stage 2. The nomenclature for the various terms of the Stage 2 calculations is illustrated on Figure 6. The proper equation is that given below; it is the solution for (k_v) with the assumption that $(k_h/k_v = 1)$.

$$K2 = R_T \, (d^2/16Lf) \, \{Ln[u(1,r_o,o)] + a \, Ln[u(1,r_o,2b_2)]\} \, Ln(H_1/H_2')/(t_2 - t_1) \qquad (3.5\text{-}5)$$

where:

f = $1 - 0.5623 \, Exp(-1.566 \, L/D)$

L = Length of Stage 2 cylinder below casing

$u(1,r_o,o)$ = $\{L/D_2 + \sqrt{1 + (L/D_2)^2}\}^2$

$u(1,r_o,2b_2)$ = $\dfrac{4b_2/D_2 + L/D_2 + \sqrt{1 + (4b_2/D_2 + L/D_2)^2}}{4b_2/D_2 - L/D_2 + \sqrt{1 + (4b_2/D_2 - L/D_2)^2}}$

D_2 = Diameter of Stage 2 extension (normally casing ID)

b_2 = Distance from <u>center</u> of Stage 2 extension to underlying boundary

And the other terms are as defined previously. The factor (f) was introduced to account for the non-convergence of the Hvorslev equations as $(L \rightarrow 0)$. For field use, the geometric terms are combined into a single constant:

$$K2 = R_T \, G2 \, Ln(H_1/H_2')/(t_2 - t_1) \qquad (3.5\text{-}6)$$

where: $G2$ = $(d^2/16Lf) \, \{Ln[u(1,r_o,o)] + a \, Ln[u(1,r_o,2b_2)]\}$

A complete example is given in the Sample Calculation, Appendix A.

3.5.3 Time-Weighted Averaging. Whether one uses the Laplacian or the Green-Ampt model for groundwater flow, there are still transient effects at the beginning of every type of field or laboratory permeability test. The observed effect is to indicate a high permeability, gradually decreasing to some relatively constant value corresponding to a steady-state flow condition. Such an effect is usually noted in the TSB. Therefore, the test must be conducted "long enough" to achieve virtually the steady-state condition or the results will be not only too high but also erratic. In addition, a single value each of (K1) and (K2) must be used in the final data reduction (Section IV).

There is no reliable method for pre-calculating the length of time required to achieve steady-state. Rather, the observational method is used. The appropriate apparent permeability (K1 or K2) is calculated for each time increment, and/or over longer periods of time; when these appear to be stable, they are checked using arithmetic time - weighted averages, e.g.,

$$K' = \Sigma\ (TiKi)/\Sigma\ (Ti) \tag{3.5-7}$$

where: K' = Arithmetic Time-Weighted Average (ATWA) Permeability

           Ti = Time Duration of Test Increment (i)

           Ki = Permeability Measured during Test Increment (i)

This is theoretically exact for a single run (between refills). Time - weighted averaging also provides a rational basis for smoothing the (often) slightly erratic individual (K1,K2) values from the various time increments. An example of time-weighted averaging is given in the Sample Calculations, Appendix A.

3.5.4 Termination Criteria. Infiltration theory indicates that the apparent permeabilities (K1,K2) should forever decrease at an ever-and-ever decreasing rate. Observations in over 200 of these tests show that a steady-state condition or a close approximation of it is achieved in reasonable testing periods. A log-log plot of apparent permeability versus time is useful in determining when steady-state is achieved. Eventually, the (K1,K2) plots fluctuate about stable values. An example of such a plot is given in the Sample Calculation, Appendix A. This plot illustrates the importance of fairly closely spaced readings at the beginning of each stage, which allow separating the long-term behavior from the short-term fluctuations, i.e., enhance the "signal-to-noise" ratio.

In most tests, time-weighted averages become quite stable, often to within 1 to 5%. A reasonable set of criteria for terminating a stage is as follows:

*   The time-weighted averages do not show an upwards or downwards trend with time,

and

*   Do not fluctuate more than 10 to 20% among themselves,

and

*   Maintain this behavior over a "sufficiently long" time, 12 - 72 hours depending on permeability.

IV. DATA REDUCTION

4.1 Basic Procedure, In some cases, the (K1') or (K2') values may be adequate for the purpose of the test. More generally, the test is performed to determine the actual (k_h, k_v). This section outlines how to convert the (K1',K2') values calculated as outlined in Paragraphs (3.5.1) and (3.5.2) into the real permeabilities (k_h, k_v). Details for the common case are covered below.

    4.1.1 Simultaneous Equations. The equations presented earlier for determining (K1,K2) are special cases of more general relationships. These more general equations define the degree of anisotropy by the parameter:

$$m = \sqrt{k_h/k_v} \qquad (4.1\text{-}1)$$

This parameter affects the geometric terms of the various equations. Each stage has its own equation with a different effect of (m). In a general sense, these can be written as:

Stage 1: $k_v = G1_m \ \mathrm{Ln}(H_1/H_2')(t_2\text{-}t_1)$
                 $K1 = G1 \ \mathrm{Ln}(H_1/H_2')(t_2\text{-}t_1)$

or $k_v = K1 \ (G1_m/G1)$ $(4.1\text{-}2)$

where: $G1_m$ = Geometric factor including (m)

Similarly, for Stage 2,

$$k_v = K2' \ (G2_m/G2) \qquad (4.1\text{-}3)$$

If the soil medium being tested is homogeneous (although cross-anisotropic and possibly bounded), the vertical permeability (k_v) must be the same in both stages. Hence, (4.1-2) and (4.1-3) provide two equations in the two unknowns $(m = \sqrt{k_h/k_v})$ and (k_v). The resulting equation is:

$$K1'(G1_m/G1) = k_v = K2'(G2_m/G2)$$

or $K2'/K1' = (G1_m/G1)(G2/G2_m)$ $(4.1\text{-}4)$

The standpipe area (A_p) cancels for each individual stage in (4.1-2 and 4.1-3), even though different (A_p) values may have been used for Stage 1 and Stage 2, and even for different portions of either stage. The actual equations for the geometric constants involving (m) are given in Paragraph (4.2).

The ratio (K2'/K1') is known from the test; the actual values introduced are the long-term time-weighted averages, (K1' and K2'). The geometric terms are also known. Therefore, Equation (4.1-4) is satisfied only for one value of (m). Due to the complex nature of (4.1-4), trial-and-error or graphical solution works best for specific problems.

4.1.2 Calculating (k_h) and (k_v). The value for (m) is obtained as outlined above in Paragraph (4.1.1). When (m) is known, (k_v) can be calculated directly from Equation (4.1-2), and, by the definition of (m) in Equation (4.1-1).

$$k_h = m^2 k_v \qquad (4.1-5)$$

4.1.3 Stage 1 Only Method. In some individual tests, the ratio (K2'/K1') is so low that Equation (4.1-4) fails to converge. Others may have so large a (K2'/K1') ratio that the permeability values are obviously in error: (k_v) is far too low and (k_h) is far too high. This is usually due to inhomogeneity of the tested material. Advancing Stage 2 into a zone of lower permeability will cause a low (K2'/K1'). Conversely, advancing into a zone of higher permeability (such as a poor lift joint in fill or a silt/sand seam in natural materials) yields a very high (K2'/K1').

These events are handled by using a conservative (m) from the best-behaved tests and introducing that value into Equation (4.1-2).

4.2 Image Equation with Smear. The basic Hvorslev equations apply most directly to masses of infinite depth and below the groundwater level. Neither test pads nor liners often meet these criteria. Therefore, results calculated by using the Hvorslev equations directly for such cases will not be correct. For a given permeability, both proximity to a drainage zone and the vertical gradient due to gravity cause the flow to be greater than the basic Hvorslev equations would predict. The basic Hvorslev equations therefore predict a higher permeability than the material really has. The vertical gradient effect can be overcome by using the head as from the top of the standpipe to the groundwater level. A method for accounting for the proximity effect and proving the previous assertion was needed.

The method of image wells has been used in geohydrology for years. The classic example is the solution for a well near a river, found in many textbooks. However, the method is not limited to two-dimensional situations such as this illustration. Any solution for an infinite or semi-infinite medium which describes the potential field (head distribution) can be converted to a solution for a finite medium bounded by a plane by using the Image Potential technique (Carslaw & Jaeger, 1959).

The basic idea is that halfway between a source and a sink of equal but opposite strength will be a plane of zero potential. So, if there is a plane of zero potential (head), its effect can be replaced by an "image" source/sink located twice as far away from the sink/source as is the midway plane. If the test (source) is set a distance (b) above the drainage blanket, the flow field will be the same as if there were no blanket but there was an image test (sink) with negative head at a distance of (2b) below the real test. Since the drainage blanket is at zero head, the head at the test is taken as the total head lost: (b) plus the excess pressure (ht) applied at the infiltration point of the test.

Consider also the case where both the real and image sources have equal strengths and both are sources (positive head) or both are sinks (negative head). By the same logic as given above, the midway plane will be a no-flow boundary, corresponding to an impermeable bottom boundary located at a depth (b) below the real test.

4.2.1 Stage 1. The Hvorslev-Image equation (Case "C") for the flush-bottomed portion of the test is given by:

$$k_v = (\pi d^2/11mD_1)[1+a(D_1/4mb_1)]Ln(H_1/H_2')/(t_2-t_1) \qquad (4.2\text{-}1)$$

where:

d	=	ID of Standpipe
D_1	=	Test diameter for Stage 1
b_1	=	Thickness of test medium below base of casing
H_1	=	Initial head ($t=t_1$)
H_2'	=	Corrected final head = H_2-c (see Paragraph 3.5)
t_2	=	Final time
t_1	-	Initial time
a	=	-1 for permeable bottom boundary
a	=	0 for infinite depth to bottom boundary
a	=	+1 for impermeable bottom boundary

Equation (4.2-1) can also be written as:

$$k_v = G1_m \, Ln(H_1/H_2')/(t_2-t_1) \qquad (4.2\text{-}2)$$
$$G1_m = (\pi d^2/11mD_1)[1+a(D_1/4mb_1)]$$

4.2.2 Stage 2. Similarly, for the cylindrical case (Hvorslev "G"), the Image equation (with sidewall smear) is that given by:

$$k_v = \quad (d^2/16Lfm^2) \, \{Ln[u(m,r_o+T,o)] \, + \, a \, Ln[u(m,r_o,2b_2)]$$
$$+ \, p \, Ln[u(m,r_o,o)/u(m,r_o+T,o)]\} \, Ln(H_1/H_2')/(t_2-t_1) \qquad (4.2\text{-}3)$$

where: L   =   Length of Stage 2 extension

f   =   1-0.5623 Exp(-1.566 L/D)

$u(m,r_o,o)$ = $[mL/D_2 + \sqrt{1+(mL/D_2)^2}]^2$

$u(m,r_a,2b_2)$ = $\dfrac{4mb_2/D_2+mL/D_2+\sqrt{1+(4mb_2/D_2+mL/D_2)^2}}{4mb_2/D_2-mL/D_2+\sqrt{1+(4mb_2/D_2-mL/D_2)^2}}$

$u(m,r_o+T,o)$ = $\{mL/(D_2+2T) + \sqrt{1+[mL/(D_2+2T)]^2}\}^2$

p   = k_b/k_s

k_s =   Permeability of smeared zone

T   =   Thickness of smeared zone (0.6cm=0.25in)

D_2 =   Diameter of Stage 2 extension

b_2 =   Distance from <u>center</u> of Stage 2 cylinder to underlying boundary

And the other terms are as defined above for Stage 1. Equation (4.2-3) can be written in the generic format as:

$$k_v = G2S \; Ln(H_1/H_2')/(t_2-t_1) \tag{4.2-4}$$

where: G2S   = $(d^2/16Lfm^2)\{Ln[u(m,r_o+T,o)] + a \; Ln[u(m,r_o,2b_2) + p \; Ln[u(m,r_o,o)/u(m,r_o+T,o)]\}$

The generic expression for (K2'/K1') as a function of the test geometry is Equation (4.1-4). Following the steps outlined in Paragraph (4.1.1),

$$K2'/K1' = (G1_m/G1) \cdot (G2/G2S) \tag{4.2-5}$$

where:

$(G1_m/G1)$ = $(1/m) [1+a(D_1/4mb_1)]/[1+a(D_1/4b_1)]$

$(G2/G2S)$ = $\dfrac{m^2\{Ln[u(1,r_o,o)]+aLn[u(1,r_o,2b_2)]}{Ln[u(m,r_o+T,o)]+aLn[u(m,r_o,2b_2)]+pLn[u(m,r_o,o)/u(m,r_o+T,o)]}$

Equation (4.2-5) is solved by taking an appropriate (p) and determining (m) by trial-and-error or by a graphical solution such as Figure 7.

The value of (p) is not determined in the test. The normal range of (p) is from 2 to 20; (p=1) indicates no smear. The following values for (p) have yielded satisfactory results, consistent with apparently non-smeared tests on the same tested units:

K2'/K1'	p
>1.1	1
0.9 - 1.1	1,2
0.8 - 0.9	2,5
0.7 - 0.8	5,10
0.6 - 0.7	10,20
0.4 - 0.6	15,20
0.4>	*

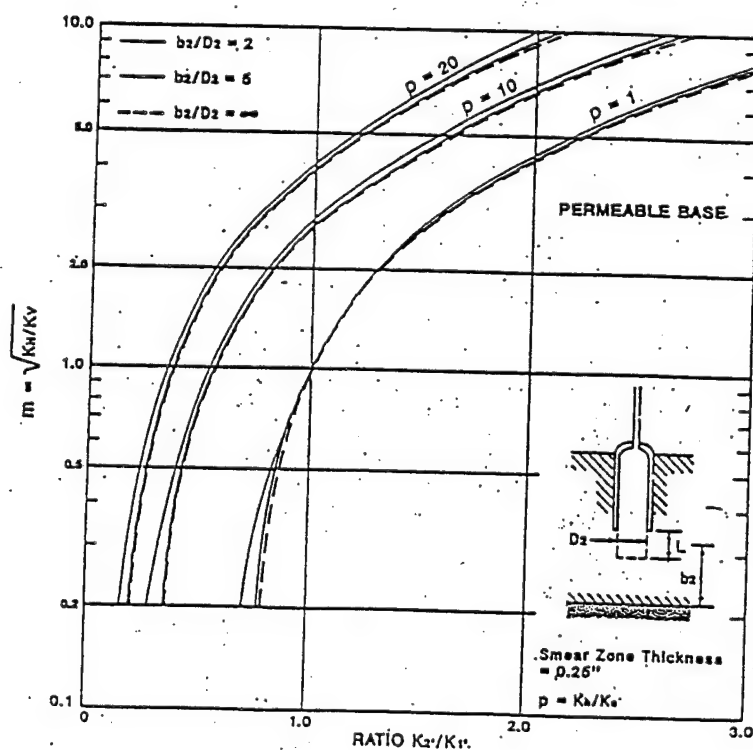* Use Stage 1 Only Approach - Paragraph (4.1.3).

FIGURE 7
GRAPH FOR (m) - (L/D=1.5)

4.3 Non-Saturated Media. Field permeability tests are frequently performed on materials which are not fully saturated. Such materials affect the tests in two ways:

* The hydraulic head is dissipated over the (changing) distance from the point of inflow into the soil to the "wetting front", where the soil is considered fully saturated (Green-Ampt Model).

* Unsaturated clays exhibit "soil suction", which effectively adds to the hydraulic head.

In the Two-Stage test, infiltration into the soil is three-dimensional. The majority of the head loss occurs close to the inflow surface, even in a fully saturated material. About 50% of the loss occurs within one test radius of the inflow surface. For a typical Two-Stage test, disregarding wetting front distance theoretically yields a permeability 10 to 50% too high.

The effect of soil suction is roughly proportional to the ratio of suction to applied head. The effect of suction alone on a permeability test can be expressed as:

$$k_a/k_t = (1 + S/h_o) \qquad (4.3\text{-}1)$$

where:
$\quad k_a \quad = \quad$ Observed permeability
$\quad k_t \quad = \quad$ True saturated permeability
$\quad S \quad = \quad$ Soil suction
$\quad h_o \quad = \quad$ Applied head

The Two-Stage test normally operates with heads 3 to 6 times those of other test methods, minimizing the relative effect of suction.

These two effects can be handled using the graph presented on Figure 8. That figure is based on numerical solutions for the equipotential surfaces in an infinite medium ($a=0$). However, for the typical real test, the dimensionless flow volume is such that the equipotentials do not vary significantly from the ellipsoids in either the permeable-base ($a=-1$) or impermeable-base ($a=+1$) cases. The actual volume, which includes an allowance for the impermeable casing, has been included on Figure 8.

When using Figure 8, the initial volume (V_o) is taken as:

* Stage 1 - The volume of a hemi-ellipsoid having the diameter (D_1) and height ($D_1/4$).

$$V_{o1} = (\pi/24) \, D_1^3 \qquad (4.3\text{-}2)$$

* Stage 2 - The volume of the Stage 2 cylinder.

$$V_{o2} = (\pi/4) \, D_2^3 \, (L/D_2) \qquad (4.3\text{-}3)$$

The term (V_w) is the total volume of water which has infiltrated into the soil through the end of each stage, allowing for that removed in the Stage 2 extension. The (n_a) term is the soil's air porosity.

Figure 8 is applied first to the individual (K1) values from Stage 1:

$$K1_t = K1/[R(1+s/H_o)] \qquad (4.3\text{-}4)$$

where: $K1_t$ = K1 corrected for suction and wetting front
R = Permeability ratio from Figure 8

Then, Figure 8 is similarly applied to the individual (K2) values from Stage 2:

$$K2_t = K2/[R(1+s/H_o)] \qquad (4.3\text{-}5)$$

where: $K2_t$ = K2 corrected for suction and wetting front

Thereafter, $(K1_t, K2_t)$ are used in Equation (3.5-7) for the average values $(K1'_t, K2'_t)$. These are then introduced into Equations (4.2-5) for (m) then (4.1-2) for (k_v), and finally (4.1-5) for (k_h).
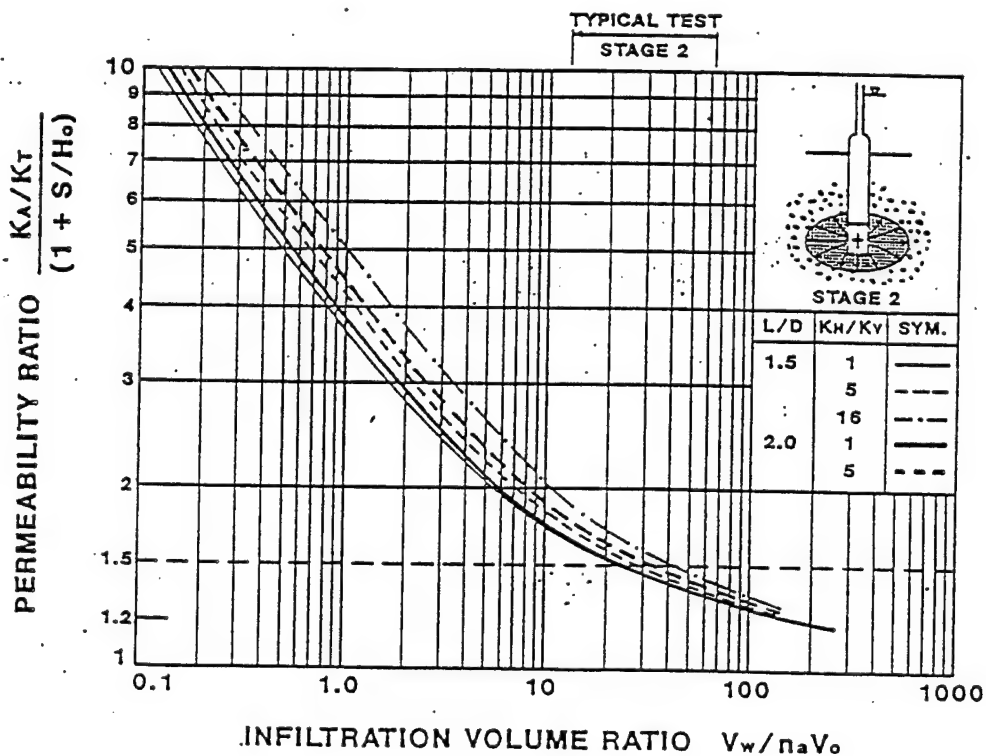

FIGURE 8
NON-SATURATION EFFECTS

V. EXPERIENCE WITH TSB TESTS

As of January, 1992, STEI alone has been involved in some 200 tests (40+ projects) on recompacted materials and 90 tests (6 projects) in natural materials. These have generally been of relatively low permeability [10(-6) to 10(-9) cm/sec]. Some conclusions from this experience are given below.

5.1 Types of Projects. The test has been successful in many types of soils:

> 5.1.1 Test Pads and Liners. It has·been used in such conditions for test units from 20 to 60 inches thick. Materials have ranged from CH-OH (Liquid Limit 100+, clay content 70%+) to SC/GC (Liquid Limit 30-, gravel content up to 30%, clay content 12%). Vertical permeabilities have been successfully measured from the mid 10(-7)'s to the low 10(-9)'s (values in cm/sec).

> 5.1.2 "Natural" Deposits. It has been very successful in·clays to depths of 10 to 15 feet. Where the clay does not make water, it has also been successful to about 20 to 25 feet. The test was moderately successful in soft, highly layered mine tailings clay at depths up to 30 feet. It has been used up to 7 feet deep in shales. Measured vertical permeabilities have been in the same ranges mentioned above.

5.2 Comparisons with Other Methods. The accuracy and lack of false negatives of the TSB can be evaluated by the comparisons with SDRI data shown on Figure 9 and with laboratory data from undisturbed samples given on Figure 10. Of the 11 known cases where both field methods were used on the same test pads/liners the mean ratio of their conductivities was 1.1 (TSB·higher). In three known cases, the TSB proved failure defects in test pads that laboratory tests did not show, indicating the TSB avoids false positives. Experience to date can be summarized as:

> 5.2.1 Recompacted Clays. The vertical permeability (k_v) as obtained from laboratory tests, the TSB, and the SDRI generally agree quite well on test pads/liners (11 cases) which have had proper CQA. The laboratory tests tend to underestimate the horizontal permeability.

> 5.2.2 Natural Clays. Comparisons have only been made with small-scale laboratory tests. In general, there is good agreement with the TSB for vertical permeability, while laboratory tests again underestimate the horizontal permeability.

5.3 Speed. As soon as the test begins, so does the question from the client, "Does it Pass?" It is usual that (k_v) must be less than some specified value (Spec), or that (k_h) must be greater than a different (Spec). Since the maximum possible value for (k_v) is (K1), as soon as (K1 < Spec), one knows the test for (k_v) must pass. Likewise, since (K2) is the minimum possible value for (k_h), if the long-term (K2) is greater than (Spec), the test for (k_h) must pass.
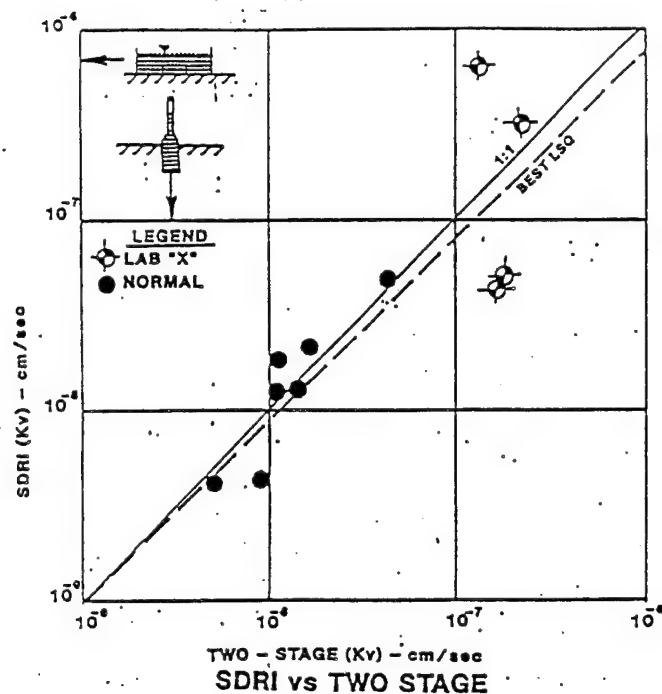
FIGURE 9
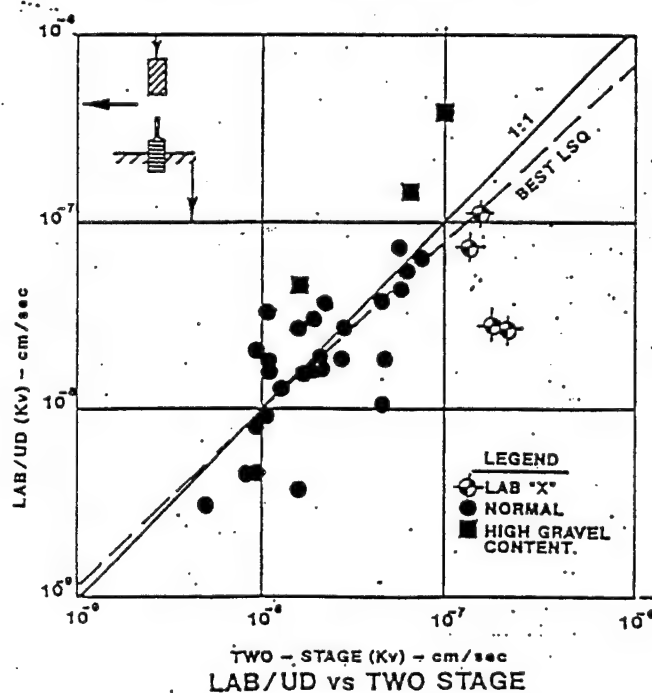TSB AND SDRI RESULTS
(Tests on Same Liners/Test Pads)

FIGURE 10
TSB AND LAB (k$_v$) RESULTS
(Tests on Same Liners/Test Pads)

Most of the TSB tests to date in test pads/liners have been for (k_v). The better the pad, i.e., the higher the $(Spec/k_v)$ ratio, the sooner $(K1 < Spec)$. In 90% of the tests where $(Spec/k_v > 1.7)$, passing was indicated in 24 hours or less. Some 75% of all tests have indicated passing within 72 hours. A marginal test unit, whose (k_v) is just below $(Spec)$, will require completing Stage 2. In general, each Stage lasts 4 to 14 days, the longer times being required to complete a test in lower permeability materials.

5.4 Volume Tested. A single typical TSB test permeates a volume around 0.6 to 1.1 cubic feet, or 60 to 200 times the volume of a typical plug tested in the laboratory (3 inch diameter, 3 inch height). The usual 5-test program thus tests about 10 to 20% the volume of an SDRI, yet yields about the same values. The TSB has a good balance of soil volume tested and speed.

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B.⊤ ⊤B

COUPLING TO FIT
AUGER ROD THREADS

0.3-0.6cm

0.3-0.6cm

$0.5-1.5 \, Da$

A ⊤ ⊤A

BEND DOWN TO ABOUT 30°
ANGLE AND SHARPEN

0.3cm

0.3cm

0.6cm

Da
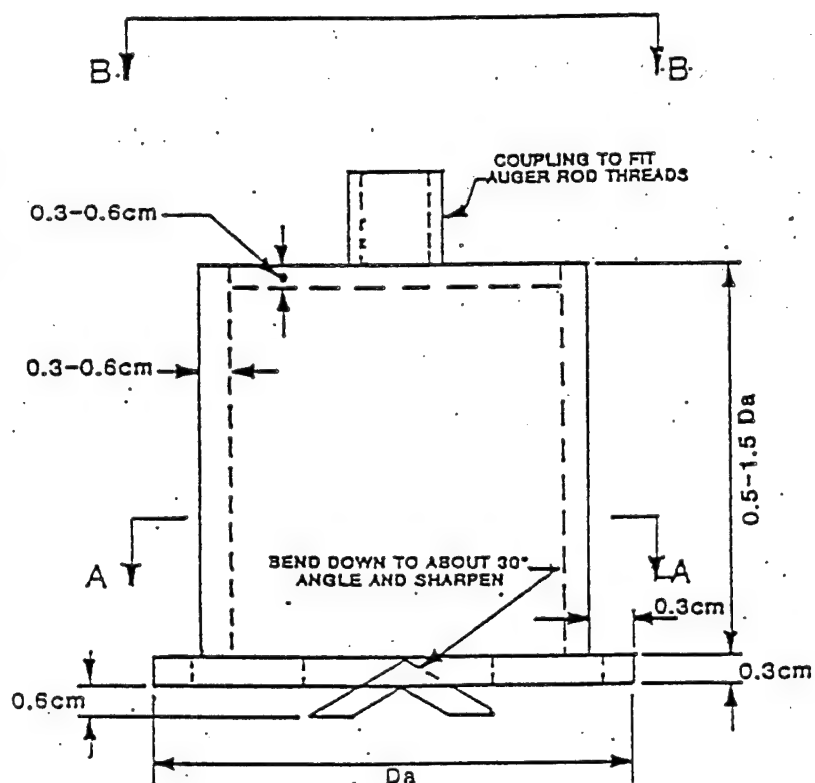
not to scale

ELEVATION
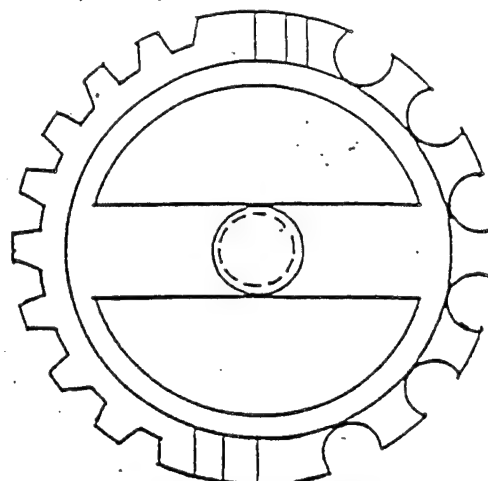
SECTION B-B

SERRATE EDGE
EITHER METHOD

SECTION A-A

NOTE: FOR FLAT AUGER, $Da = D + 5 \, cm$
      FOR REAMER, $Da = D - 0.1 \, cm$

FLAT BOTTOM REAMING AUGER

STAGE 1 CALCULATIONS

d = 1.27 cm
D = 11.43 cm
Z = 61.0 cm
R_A = 22.9 cm
b_1 = 61.0 cm

$\Delta V = 1.27 \, \Delta h_e$

$H = 144.8 + R$

$K_1 = 0.03841 \dfrac{Ln(H_1/H_2)}{t_2 - t_1}$

Ground Elev.: _____
TOC Elev.: _____ (before)
_____ (after)
Site Coords: _____

Project: _____
File No.: _____
Test No.: _____
Page: _____

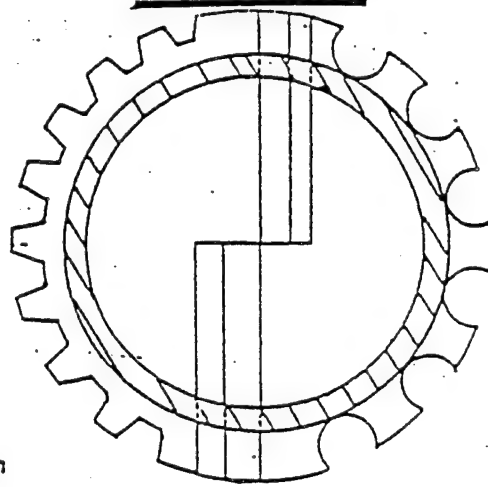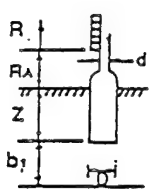| Date	Time	Δt (hrs.)	Δt (sec.)	R (cm)	H1 (cm)	H2 (cm)	K1 (cm/sec)	C (cm)	H2' (cm)	K1' (cm/sec)	Temp. (°C)	Rt Factor	K1't (cm/sec)	Cum Vol. (cc)	Cum. Hrs.	Remarks
8/01	0800	—	—	64.8	—	209.6	—	0.0	—	—	21	—	—	0.0	0.0	START
	0830	0.50	1800	47.8	209.6	192.6	1.80E-06	0.0	192.6	1.80E-06	21	0.97	1.76E-06	21.5	0.5	
	0900	0.50	1800	36.3	192.6	181.1	1.31E-06	0.0	181.1	1.31E-06	21	0.97	1.27E-06	36.0	1.0	
	1000	1.00	3600	19.7	181.1	164.5	1.03E-06	0.2	164.3	1.04E-06	22	0.96	9.99E-07	57.1	2.0	End Run
	1001	—	—	66.7	—	211.5	—	0.0	—	—	22	—	—	—	2.0	Refill
	1200	1h+59m	7140	47.0	211.5	191.8	5.26E-07	0.6	191.2	5.44E-07	23	0.94	5.09E-07	82.0	4.0	
						etc.										
8/05	1700			50.0		194.8					26				105.0	
8/06	0800	15.00	54000	38.1	194.8	182.9	4.49E-08	-2.8	185.7	3.39E-08	19	0.94	3.20E-08	205.5	120.0	STOP

STAGE 2 CALCULATIONS
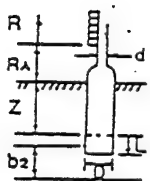
d = 1.27 cm
D = 10.16 cm
Z = 61.0 cm
R_A = 22.9 cm
b_2 = 53.3 cm

L = 15.2 cm

$\Delta V = 1.27 \, \Delta h_e$

$H = 144.8 + R$

$K_2 = 0.01570 \dfrac{Ln(H_1/H_2)}{t_2 - t_1}$

Ground Elev.: _____
TOC Elev.: _____ (before)
_____ (after)
Site Coords: _____

Project: _____
File No.: _____
Test No.: _____
Page: _____

Date	Time	Δt (hrs.)	Δt (sec.)	R (cm)	H1 (cm)	H2 (cm)	K2 (cm/sec)	C (cm)	H2' (cm)	K2' (cm/sec)	Temp. (°C)	Rt Factor	K2't (cm/sec)	Cum Vol. (cc)	Cum. Hrs.	Remarks
8/06	0930	—	—	65.4	—	210.2	—	—	—	—	20	—	—	0.0	0.0	START
	1000	0.5	1800	59.4	210.2	204.2	2.52E-07	0.0	204.2	2.52E-07	21	0.99	2.49E-07	7.6	0.5	
	1030	0.5	1800	54.3	204.2	199.1	2.21E-07	+0.2	198.9	2.28E-07	22	0.96	2.19E-07	14.1	1.0	
	1130	1.0	3600	45.7	199.1	190.5	1.92E-07	+0.3	190.2	2.00E-07	23	0.94	1.87E-07	24.9	2.0	
	1330	2.0	7200	33.3	190.5	178.1	1.47E-07	+1.3	176.8	1.62E-07	24	0.91	1.48E-07	10.6	4.0	
						etc.										
8/09	0730			21.3		166.1					16				70.0	
	1600	8.5	30600	9.2	166.1	154.0	3.88E-08	+1.1	152.9	4.25E-08	19	1.07	4.56E-08	267.9	78.5	STOP

FIGURE 3.

APPENDIX A

SAMPLE CALCULATION
FOR
2-STAGE FIELD PERMEABILITY TEST

NOTE: This is an idealized case
exhibiting virtually perfect behavior,
and is not to be considered
representative of field behavior.

JOB _SAMPLE – "A"_

SHEET NO. _1_ OF _____

CALCULATED BY _____ DATE _08 Aug 9_

CHECKED BY _____ DATE _____

SCALE _____

SAMPLE 2-STAGE CALCULATION – HYPOTHETICAL TEST "A"

1. TEST SETUP
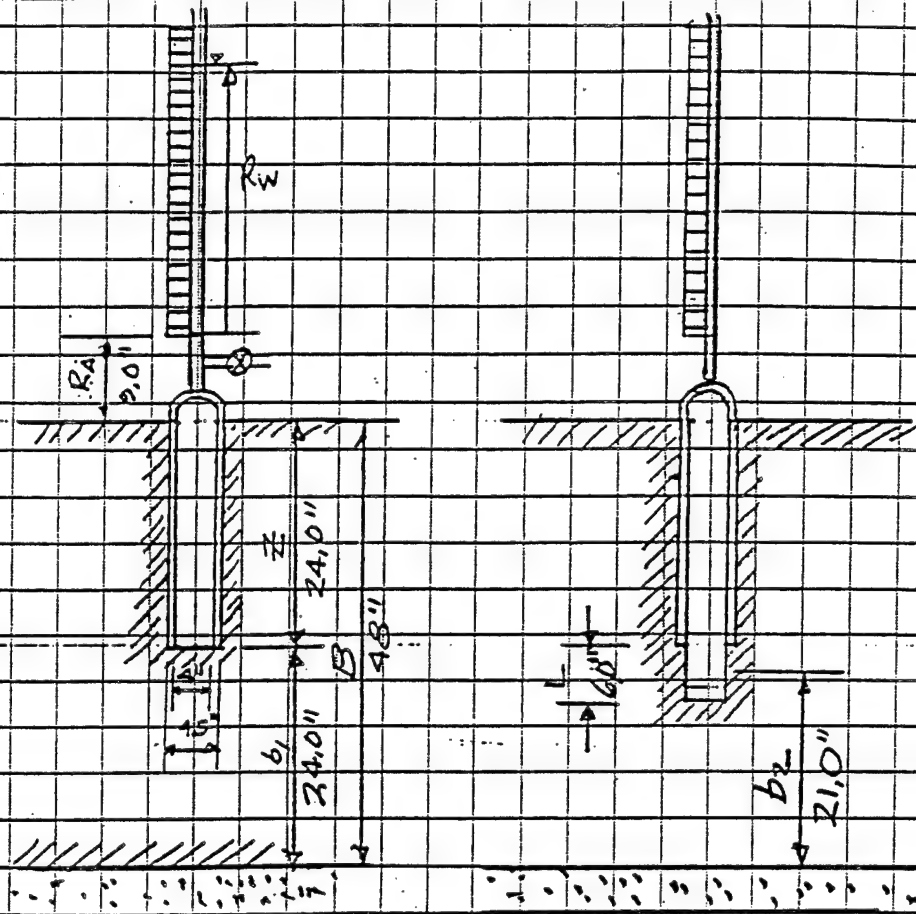
STAGE 1   STAGE 2

Note: Water seeped into casing
during bentonite sealing
Therefore, use casing
OD = 4.5" for
Stage 1 Only.

TO AVOID HYDRAULIC FRACTURING:

HEAD < 1.25 × OVERBURDEN AT CASING BOTTOM

$62.4 \frac{LB}{FT^3} H_{FT} < 1.25 \times 2.0_{FT} \times 124.8 \, ^{LB}/_{FT^3}$

$H_{FT} < 5.0 \, FT$

OR $R_W < 5.0 - 2.0 - 0.75 = 2.25 \, FT = 27.0 \, ^{IN}$

STEI 16A

SOIL TESTING ENGINEERS, INC.
316 Highlandia Drive
P.O. Box 83710
BATON ROUGE, LOUISIANA 70884
(504) 752-4790

JOB _SAMPLE — A_
SHEET NO. _2_ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE _____

2. Test Data

STAGE 1 DATA

Date	Time	RW (in.)	ØC (in.)	Temp. (°F)	TEG (in.)
8/01	08:00	25.50	—	70	13.00
	0830	18.81	0	70	13.00
	0900	14.31	0	70	13.00
	1000	7.75	+0.06	72	13.06
Rd.	1001	26.25	—	72	13.06
	1200	18.50	+0.25	74	13.31
	1600	13.88	+1.00	80	14.31
Rd.	1601	26.00	—	80	14.31
8/02	0800	15.81	-1.50	65	12.81
	1700	14.06	+1.31	74	14.12
8/03	0800	9.06	-1.25	68	12.87
	1600	8.38	+1.06	76	13.93
8/04	0800	3.75	-1.44	62	12.49
Rd.	0801	26.50	—	62	12.49
	1701	24.88	+0.50	71	13.00
8/05	0700	20.88	-0.62	66	12.37
	1700	19.69	+1.25	78	13.62
8/06	0800	15.00	-1.12	67	12.50

STAGE 2 DATA

Date	Time	RW (in.)	ØC (in.)	Temp. (°F)	TEG (in.)
8/06	0930	25.75	—	68	12.56
	1000	23.39	0.00	70	12.56
	1030	21.38	+0.06	72	12.62
	1130	18.00	+0.13	74	12.75
	1330	13.12	+0.50	76	13.25
	1700	7.12	+0.69	80	13.94
R	1701	26.56	—	80	13.94
8/07	0700	6.06	-1.12	70	12.82
R	0701	25.25	—	70	12.82
	1200	19.88	+0.31	74	13.03
	1700	15.94	+0.56	78	13.59
8/08	0800	3.81	-1.50	62	12.09
R	0802	26.31	—	62	12.09
	1700	20.56	+1.69	74	13.78
8/09	0730	8.38	-1.56	60	12.22
	1600	3.62	+0.44	66	12.66

READ FROM TEG

CALCULATED FROM TEG LEVELS

READ FROM TEG

Calculate from TEG Levels

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JOB _SAMPLE — A_

SHEET NO. _3_ OF ___

CALCULATED BY ___ DATE ___

CHECKED BY ___ DATE ___

SCALE ___

3. STAGE 1

A. CALCULATE FACTOR [Eq. 3.5-4]

$$K1 = F_1 \frac{Ln(H_1/H_2)}{t_2 - t_1} \quad \text{where } F_1 = \frac{\pi d^2}{t_1 D}\left(1 - \frac{D}{4b_1}\right)$$

$d = 0.50'' = 1.27$ cm   (See sketch on p.1)
$D = 4.50'' = 11.43$ cm   (See note on p.1)
$b_1 = 24.0'' = 60.96$ cm   (See sketch on p.1)

$$F_1 = \frac{\pi (1.27)^2}{11 \times 11.43}\left(1 - \frac{11.43}{4 \times 60.96}\right)$$

$$\boxed{F_1 = 0.03841 \text{ cm}}$$

B. CALCULATIONS

1. K1 $H1 = B_1 + R_A + R_{W1}$ = 48.00 + 9.00 + R_{W1} = 57.00 + R_{W1}
   $H2' = B + R_A + R_{W2} - C$ = 48.00 + 9.00 + ($R_{W2} - C$) = 57.00 + R_{W2}

i. Period 1700 08/02 — 0800 08/03

$$K1 = .03841 \frac{Ln[(57.00 + R_{W01})/(57.00 + R_{02} + C)]}{\Delta t \text{ (sec)}}$$

$H1 = 57.00 + 14.06 = 71.06$ Note: TEG cell 1.25"
$H2' = 57.00 + 9.06 + 1.25 = 67.31$ in this period. ∴ 1.25"
                                      of the Test drop is purely temp.

$\Delta t = 15 hrs \times 3600 \text{ s/hr} = 54000$ sec.

$$K1_c = 0.03841 \text{ cm} \frac{Ln(71.06/67.31)}{54000 \text{ sec}} = 3.86 \times 10(-8)$$

Now, apply viscosity correction
$Temp = \frac{1}{2}(74 + 68) = 71°$
From chart, viscosity Factor = 0.96 = R_T

$$K1_t = R_T \cdot K1_C = 0.96 \times [3.86 \times 10(-8)] = 3.71 \times 10(-8) \frac{cm}{sec}$$

See p.6 for Tabulation of all points

STEI 16A

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JOB _SAMPLE — H_
SHEET NO. _4_ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE _____

4, STAGE 2

A. CALCULATE FACTOR [Eq. 3.5-5, with $a = -1$]

$$K_2 = F \frac{Ln(H_1/H_2)}{t_2 - t_1}$$

where: $F = \left(\frac{1}{f}\right)\left(\frac{d^2}{16 L}\right) L_N \left[\frac{U(1, 0, 0)}{U(1, r_0, 2b_2)}\right]$

$f = 1 - 0.5623 e^{-1.566(L/D)}$

L = Extension below Casing (6 inches = 15.24 cm)

D = Borehole diameter = ID of Casing (4 inches)

$r_0 = D/2$

$b_2 = B - Z + L/2 = 48.00" - 24.00" - 6.00"/2 = 21.00"$

$$U(1, r_0, z) = \frac{(z/D + L/D) + \sqrt{1 + [z/D + L/D]^2}}{z/D - L/D + \sqrt{1 + [z/D - L/D]^2}}$$

∴ $f = 1 - 0.5623 e^{-1.566(1.5)}$ $L/D = 6"/4" = 1.5$

   $= 0.9463$

$U(1, r_0, 0) = U(1, D, 0) = \frac{(0 + 1.5) + \sqrt{1 + [0 + 1.5]^2}}{(0 - 1.5) + \sqrt{1 + [0 - 1.5]^2}} = \frac{1.5 + \sqrt{1 + 1.5^2}}{-1.5 + \sqrt{1 + 1.5^2}}$ $[U(1, r_0, 0)$ Eq. 3.5-5]

          $= 10.9083$ $= [1.5 + \sqrt{1 + 1.5^2}]^2$

$2b_2 = 2 \times 21 = 42"$

$U(1, r_0, 2b_2) = U(1, D, 42) = \frac{(84/4 + 1.5) + \sqrt{1 + [84/4 + 1.5]^2}}{(84/4 - 1.5) + \sqrt{1 + [84/4 - 1.5]^2}} = \frac{22.5 + \sqrt{1 + 22.5^2}}{19.5 + \sqrt{1 + 19.5^2}}$

          $= 1.1537$ $[U(1, r_0, 2b_2)$ Eq. 3.5-5]

∴ $F = \frac{1}{0.9463} \times \frac{1.27^2 \ cm^2}{16 \times 15.24 cm} L_N (10.9083 / 1.1537)$

   $\boxed{F = 0.01570 \ cm}$

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JOB **SAMPLE — A**
SHEET NO. **5** OF
CALCULATED BY DATE
CHECKED BY DATE
SCALE

$H_1' = 57.00 + R_{in_1}$ $H_2' = 57.00 + (R_{in_2} - C)$

through 17:00 8/08

$= 83.31 = 83.31$ in
$= -1.69 = 75.87$ in

Notes: During this period the TEG rose 1.69" due only to temp. ∴ subtract 1.69" to get corrected level in Test

minutes $= 8(3600) + 58(60) = 32280$ sec.

$(in H_2 O)$

$$\frac{LN(83.31/75.87)}{32280 \text{ sec}}$$

$(4.55 \times 10^{(-8)})$ cm/sec

$= \frac{1}{2}(62 + 74) = 68.0°$

viscosity factor $= 1.00 = R_T$

$K = 1.00 \times [4.55 \times 10^{(-8)} \text{ cm/sec}] = 4.55 \times 10^{(-8)}$ cm/sec.

to Tabulation of all data points

EXPL-1.XLS

SAMPLE - A
STAGE 1

<div style="text-align:right">p. 6</div>

DATE	TIME	READING (in)	TIME (sec)	H1 (in)	H2 (in)	K1 (cm/sec)	TC (in)	H2' (in)	K1c (cm/sec)	Temp	Temp Factor	K1tc (cm/sec)	Cum. Hrs.	Cum. Volume (cu. cm)	Remarks
8/1	8:00	25.50	-	-	82.50	-	0.00	82.50	-	70	-	-	0.0	0.0	START
	8:30	18.81	1800	82.50	75.81	1.80E-06	0.00	75.81	1.80E-06	70	0.97	1.76E-06	0.5	21.5	
	9:00	14.31	1800	75.81	71.31	1.31E-06	0.00	71.31	1.31E-06	70	0.97	1.27E-06	1.0	36.0	
	10:00	7.75	3600	71.31	64.75	1.03E-06	-0.06	64.69	1.04E-06	72	0.96	9.99E-07	2.0	57.1	end run
	10:01	26.25	60	64.75	83.25	-	0.00	83.25	-	72	-	-	2.0	57.1	Refill
	12:00	18.50	7140	83.25	75.50	5.26E-07	-0.25	75.25	5.44E-07	74	0.94	5.09E-07	4.0	82.0	
	16:00	13.88	14400	75.50	70.88	1.68E-07	-1.00	69.88	2.06E-07	80	0.89	1.84E-07	8.0	96.9	end run
	16:01	26.00	60	70.88	83.00	-	0.00	83.00	-	80	-	-	8.0	96.9	Refill
8/2	8:00	15.81	57540	83.00	72.81	8.74E-08	1.50	74.31	7.38E-08	65	0.94	6.96E-08	24.0	129.7	
	17:00	14.06	32400	72.81	71.06	2.88E-08	-1.31	69.75	5.09E-08	74	0.98	4.99E-08	33.0	135.3	
8/3	8:00	9.06	54000	71.06	66.06	5.19E-08	1.25	67.31	3.86E-08	68	0.96	3.71E-08	48.0	151.4	
	16:00	8.38	28800	66.06	65.38	1.38E-08	-1.06	64.32	3.56E-08	76	0.95	3.38E-08	56.0	153.6	
8/4	8:00	3.75	57600	65.38	60.75	4.90E-08	1.44	62.19	3.34E-08	62	0.99	3.29E-08	72.0	168.5	end run
	8:01	26.50	60	60.75	83.50	-	0.00	83.50	-	62	-	-	72.0	168.5	Refill
	17:01	24.88	32400	83.50	81.88	2.32E-08	-0.50	81.38	3.05E-08	71	1.02	3.11E-08	81.0	173.7	
8/5	7:00	20.88	50340	81.88	77.88	3.82E-08	0.62	78.50	3.22E-08	66	0.99	3.20E-08	95.0	186.6	
	17:00	19.69	36000	77.88	76.69	1.64E-08	-1.25	75.44	3.40E-08	78	0.95	3.22E-08	105.0	190.4	
8/6	8:00	15.00	54000	76.69	72.00	4.49E-08	1.12	73.12	3.35E-08	67	0.94	3.20E-08	120.0	205.5	STOP

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JOB _SAMPLE – A_
SHEET NO. _9_ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE _____

5.B. Calculation for K2' (See p.7 for the data points)

This is virtually the same calculation as on p.8

Period (hrs)	Calculation		K2'
0.0–2.0	$\dfrac{24.5\times1.80+21.9\times1.80+18.7\times3.60}{1.80+1.80+3.60}$	$=\dfrac{151.56}{7.20}=21.05\times10(-8)$	$2.11\times10(-7)$
2.0–21.5	$\dfrac{14.8\times7.20+11.0\times12.60+7.52\times50.34}{7.20+12.60+50.34}$	$=\dfrac{623.72}{70.14}=8.89\times10(-8)$	$8.89\times10(-8)$
21.5–31.5	$\dfrac{5.94\times17.94+4.14\times18.00}{17.94+18.00}$	$=\dfrac{191.88}{35.94}=5.34\times10(-8)$	$5.34\times10(-8)$
31.5–55.5	$\dfrac{1.46\times54.00+4.55\times32.28}{54.00+32.28}$	$=\dfrac{387.11}{86.28}=4.49\times10(-8)$	$4.49\times10(-8)$
55.5–78.5	$\dfrac{4.49\times52.20+4.56\times30.60}{52.20+30.60}$	$=\dfrac{373.91}{82.80}=4.52\times10(-8)$	$4.52\times10(-8)$

The last two periods are virtually identical and last some 24 hours each (appropriate for this permeability land)

ie K2' $=\dfrac{387.11+373.91}{86.28+82.80}=\dfrac{761.62}{169.08}=4.50\times10(-8)$

$$\boxed{K2' = 4.50\times10^{-8}\ cm/sec}$$

6.0 Calculate Kv and Kh

A. Smear.
Check by K2/K1 ratio needed for next step

$K2'/K1' = \dfrac{4.50\times10(-8)}{3.21\times10(-8)} = 1.40$

Probably no smear since > 1.0

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JOB _____
SHEET NO. 10 R   OF _____
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CHECKED BY _____ DATE ____
SCALE _____

B. EQUATION   (FOR SHORTCUT, SEE p. 12)

$$K_2'/K_1' = \left(\frac{G_1 m}{G_1}\right)\left(\frac{G_2}{G_2 S}\right) \qquad [Eq. 4.2-5]$$

where $\quad G_1 m/G_1 = (1/m)[1 + a(D_i/4mb_1)]/[1 + a(D_1/4b_1)]$

but, $D_1 = 11.43$ cm (p.3) $a = -1$ (Perm. Base

$b_1 = 20.96$ cm (p.3)

$$G_1 m/G_1 = (1/m)\left[1 - \frac{11.43}{4(m)(20.96)}\right]/\left[1 - \frac{11.43}{4(20.96)}\right]$$

$$= (1/m)(1/.864)(1 - \frac{1}{21.33m})$$

$$G_2/G_2 S = \frac{m^2 \{Ln[u(1,r_0,0)] + a\, Ln[u(1,r_0,2b_2)]\}^?}{Ln[u(m,r_0+,0)] + a\, Ln[u(m,r_0,2b_2)] + \beta\, Ln[u(m,r_0,0)/u(m,.}$$

$$a = -1$$
$$\beta = 1$$

Thus $G_2/G_2 S = \dfrac{m^2\, Ln[u(1,r_0,0)/u(1,r_0,2b_2)]}{Ln[u(m,r_0,0)/u(m,r_0,2b_2)]}$

$$u(1,r_0,0) = \left[4/D + \sqrt{1+(4/D)^2}\right]^2 = \left[1.5 + \sqrt{1+1.5^2}\right]^2 = 10.90833$$

$$u(1,r_0,2b_2) = \frac{4b_2/D_2 + 4/D_2 + \sqrt{1+(4b_2/D_2 + 4/D_2)^2}}{4b_2/D_2 - 4/D_2 + \sqrt{1+(4b_2/D_2 - 4/D_2)^2}}$$

$b_2 = 21"$ (See p.1)
$D_2 = 4.0"$ (See p.1

$$= \frac{4(21/4) + 6/4 + \sqrt{1+(4 \times 21/4 + 6/4)^2}}{4(21/4) - 6/4 + \sqrt{1+(4 \times 21/4 - 6/4)^2}} = \frac{45.0222}{39.0256}$$

$$u(1,r_0,2b_2) = 1.15366$$

$$G_2/G_2 S = \frac{m^2\, Ln[10.90833/1.15366]}{Ln[u(m,r_0,0)/u(m,r_0,2b_2)]} = \frac{2.24659\, m^2}{Ln[u(m,r_0,0)/u(m,r_0,2b_2)]}$$

$$K_2'/K_1' = \frac{1}{m}\cdot\frac{1}{.864}(1 - \frac{1}{21.33m}) \cdot \frac{2.24659\, m^2}{Ln[u(m,r_0,0)/u(m,r_0,2b_2)]}$$

$$= 2.3738\, m (1 - \frac{1}{21.33m}) \frac{1}{Ln[u(m,r_0,0)/u(m,r_0,2b_2)]}$$

STEI 16A

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JOB SAMPLE — A
SHEET NO. 11 R   OF
CALCULATED BY   DATE
CHECKED BY   DATE
SCALE

(i) Try $m = 2.0$

$$U(m, r_2, 0) = \left[mL/D_2 + \sqrt{1 + (mL/D_2)^2} \right]^2$$
$$= \left[1.5m + \sqrt{1 + (1.5m)^2} \right]^2$$
$$U(2, r_2, 0) = \left[1.5 \times 2 + \sqrt{1 + (1.5 \times 2)^2} \right]^2 = 6.1623^2$$
$$= 37.9737$$

$$U(m, r_2, 2b_2) = \frac{4mb_2/D_2 + mL/D + \sqrt{1 + (4mb_2/D_2 + mL/D_2)^2}}{4mb_2/D_2 - mL/D + \sqrt{1 + (4mb_2/D_2 - mL/D_2)^2}}$$
$$= \frac{4m \cdot 21/4 + m \cdot 6/4 + \sqrt{1 + (4m \cdot 21/4 + m \cdot 6/4)^2}}{4m \cdot 21/4 - m \cdot 6/4 + \sqrt{1 + (4m \cdot 21/4 - m \cdot 6/4)^2}}$$
$$= \frac{22.5m + \sqrt{1 + (22.5m)^2}}{19.5m + \sqrt{1 + (19.5m)^2}}$$

$$U(2, r_2, 2b_2) = \frac{22.5 \times 2 + \sqrt{1 + (22.5 \times 2)^2}}{19.5 \times 2 + \sqrt{1 + (19.5 \times 2)^2}} = \frac{90.0111}{78.0428} = 1.1538$$

$$K_2'/K_1' = 1.40 \overset{?}{=} 2.3738(2.0)\left(1 - \frac{1}{21.33 \times 2}\right)\frac{1}{Ln\left[\frac{37.9737/1.1538}{1}\right]} \cdot \frac{1}{3.4938}$$
$$= 4.7476 (.9766) \frac{1}{3.4938}$$
$$1.40 > 1.327$$

∴ need larger m

ii. Try $m = 2.4$

$$U(2.4, r_d, 0) = \left[1.5 \times 2.4 + \sqrt{1 + (1.5 \times 2.4)^2} \right]^2 = 53.8214$$

$$U(2, r_2, 2b_2) = \frac{22.5 \times 2.4 + \sqrt{1 + (22.5 \times 2.4)^2}}{19.5 \times 2.4 + \sqrt{1 + (19.5 \times 2.4)^2}} = 1.1538$$

$$K_2'/K_1' = 1.40 \overset{?}{=} 2.3738(2.4)\left[1 - \frac{1}{21.33 \times 2.4}\right] \frac{1}{Ln\left[53.8214/1.1538\right]}$$
$$= 5.6971(.9805)\left(1/3.8426\right)$$
$$1.40 < 1.454$$
∴ need smaller m

Successive trial-and-error yields $m = 2.27$

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JOB _____
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CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE _____

C. CALCULATE K_v, K_h

Note: One can avoid the calculations of pp. 10 and 11 by using the attached nomograph (Fig. 7 - Text)

The easiest calculation is using the Stage 1 equation:

$$K_v = \frac{Gtm}{Gt_i} \cdot K1'_i = \qquad [Eq. 4.1-2]$$

$$= \frac{1}{m}\left[\frac{1 - D_1/4mb_1}{1 - D_1/4b_1}\right] K1'_i$$

with $D_1 = 4.50''$

$b_1 = 24.0''$

$m = 2.27$

$$K_v = \frac{1}{2.27}\left[\frac{1 - \frac{4.5}{4(2.27)(24)}}{1 - \frac{4.5}{4(24)}}\right] \times \left[3.21 \times 10(-8)\right]$$

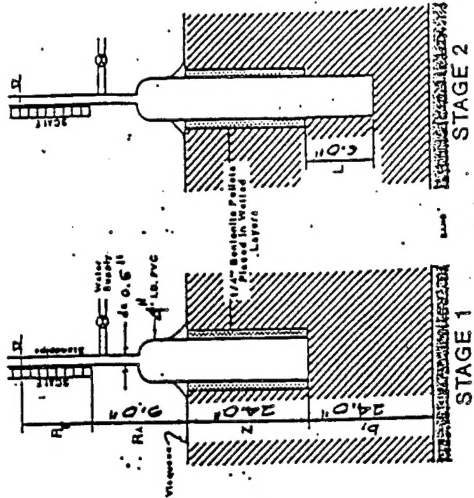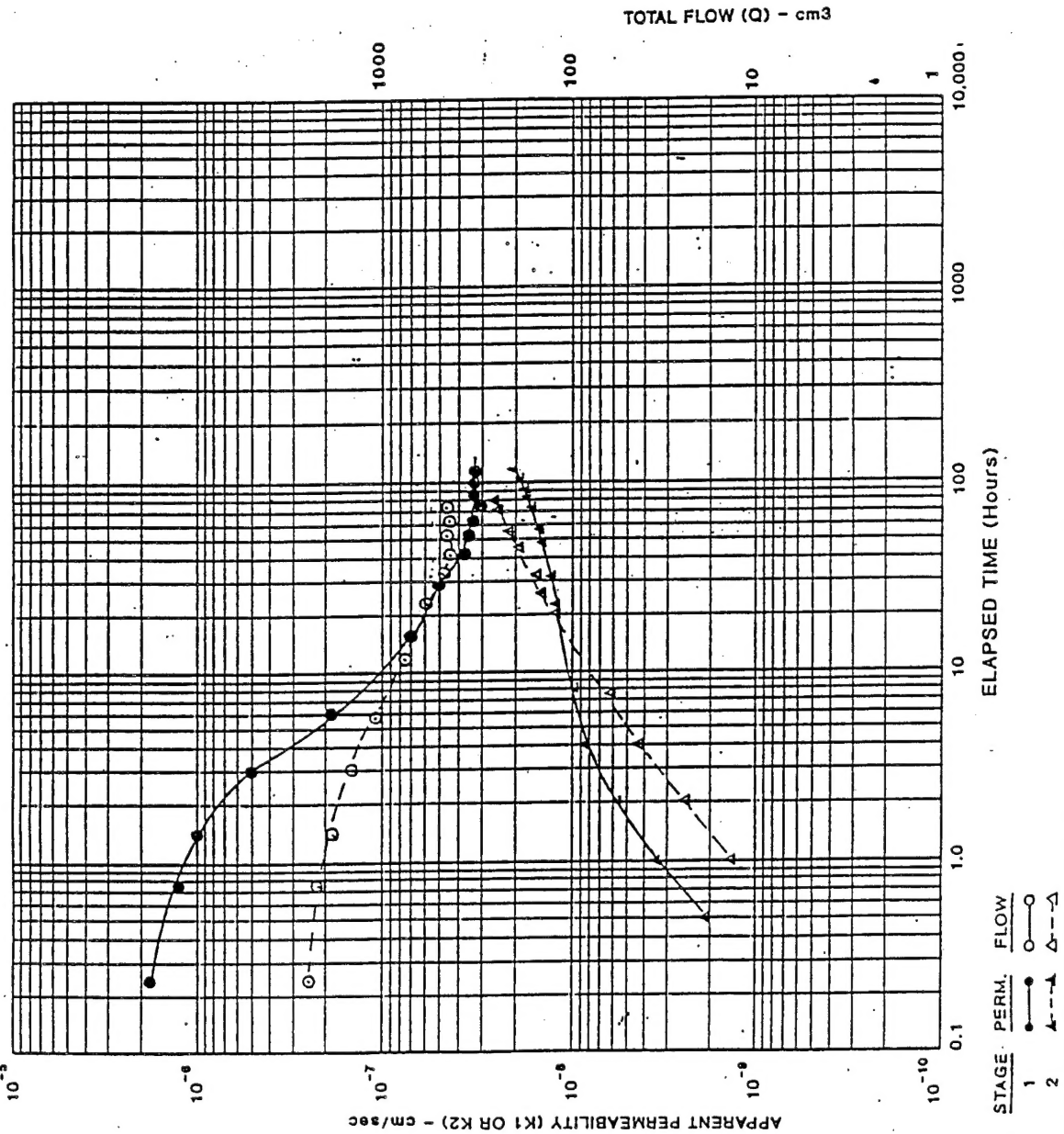$$= \frac{1}{2.27}\left[\frac{.9762}{.9464}\right] 3.21 \times 10(-8)$$

$$\boxed{K_v = 1.45 \times 10(-8) \quad cm/sec}$$

$$K_h = m^2 K_v \qquad [Eq. 4.1-5]$$
$$= 2.27^2 \left[1.45 \times 10(-8)\right]$$

$$\boxed{K_h = 7.47 \times 10(-8)}$$

STEI 16A

TWO – STAGE
FIELD PERMEABILITY TEST
SUBBASE TEST PAD

SAMPLE TEST "A"
EREWHON, CALIVARIA

GEOTECHNICAL SERVICES BY
SOIL TESTING ENGINEERS, INC.

	Baton Rouge, LA	Lake Charles, LA	
Project Engineer:	Drawn by:	Checked by:	
G.P. BOUTWELL	SJT	C.N.TSAI	
File No.:	Date:		Figure No.:
91-081			

Title: BEHAVIOR OF TEST FP- "A"

STAGE 1

STAGE 2

TOTAL FLOW (Q) – cm3

ELAPSED TIME (Hours)

APPARENT PERMEABILITY (K1 OR K2) – cm/sec

STAGE	PERM.	FLOW
1	●—●	○—○
2	▲--▲	△--△

NOTE: TUBE SIZES CAN BE ADJUSTED 1/4" TO 1"

FOIL WITH PIN-PRICK HOLE

90° ELBOW

30" - 36" ALUMINUM SCALE (MIN. 1/8" DIVISIONS) FIX TO TUBE WITH DUCT TAPE OR CLEAR STRAPPING TAPE.

1/2" OR 3/4" I.D. CLEAR SCHED. 40 PVC TUBE (30"-36")

Scale

MEASURING STANDPIPE

WATERTIGHT FITTING FOR THERMOMETER OR THERM-COUPLE, TEG ONLY.

GLUE JOINT

MALE/FEMALE PVC FITTING 1/2" TO TUBE SIZE

SCHEDULE 40 PVC DOMED CAP

PVC VALVE-FITTING TO 1/2" TUBING

LEAVE FLUSH

"O" RING

SCHEDULE 40 PVC CASING

DRILL & TAP 1/2"

SEAL JOINT WITH PFTE PIPE TAPE AND SILICONE GREASE (FAN BELT LUBRICANT)

TO BE GROUTED

NATURAL SOIL

TYPICAL PERMEAMETER